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Hudson River Road Built on Mountain Side

By Eugene Geduldiger*

An important link has been added to Route No. 3 of the New York State highway system by the completion of the Nyack-Rockland Lake road, the work on which was described in *Engineering News*, Nov. 25, 1915. This is on the main thoroughfare between New York and Albany along the west shore of the Hudson River, and passes through the River divisions of the Palisades Interstate Park, including the ever growing popular playground at Bear Mountain. The Nyack-Rockland Lake road begins at Main St. in the village of Nyack, Rockland County, and runs north along Midland Ave., then west and northwest over Hook Mountain to the southern end of Rockland Lake, a distance of 2.95 mi. It is a characteristic mountain road in that it has many turns and changing grades. The maximum grade is 9%, but this is only for a short distance, near Sta. 110. The most interesting section of the road is between Sta. 95 and 112, where the new line follows up along the rocky slopes of Hook Mountain, locally known as Goat Hill (see Figs. 1 and 2). This new line eliminates a "hair-pin" turn at Sta. 113+50 of the main line, which was very dangerous because it occurred on a heavy grade where it is impossible for a modern large automobile to turn without backing.

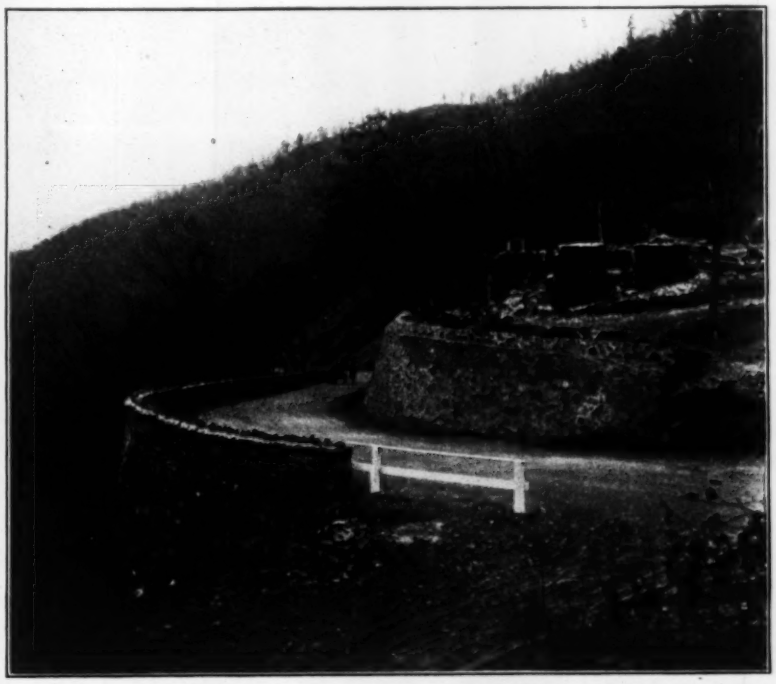

FIG. 1. NYACK-ROCKLAND LAKE ROAD; NEW LOCATION AT STA. 112

When the improvement of the road was first considered, the intention was to follow the old alignment, widening and building some kind of plateau to ease this turn. At that time it was considered impossible and impracticable to locate a highway along the dangerous side hill of talus, owing to the excessive cost of construction. After careful investigation, however, it was found that such a line would be possible and practicable.

A comparative estimate showed that the cost of locating the line along the mountain would not be excessive, and that this relocation would be nearly 500 ft. shorter than the old road besides eliminating the "hair-pin" turn. The additional cost of about $4,500 was considered small in comparison with the benefit derived. The new line begins to ascend the mountain side at Sta. 95 with a slight curve to the west and rises on an average of 8% with easy curves to Sta. 111 and Sta. 112, where, with a moderately sharp curve, it passes through a 20-ft. rock cut and meets the old location. The greater part of the location runs through a nest of trap-rock boulders that vary in size from ½ cu.ft. to 1½ cu.yd. Underlying this talus is a thin layer of earth covering a rock ledge of steep inclination. From Sta. 105+50 to Sta. 111+50 a dry wall was built on the east side of the road. This wall varies in height from 5 to 35 ft., with a base width of nearly 50% of the height.

The upper 18 in. of the wall is laid in cement mortar to afford a foundation for an 18 in. (wide) by 3 ft. 6 in. (high) wet-masonry parapet wall finished with a 6-in. rustic coping. The foundation of the wall was carried down to the rock ledge, which in some places was nearly 6 ft. below the original surface. The rock ledge was benched in order to give the wall a secure footing. The greatest part of the fill was laid in courses of rock, thereby relieving the wall of excessive pressure. From Sta. 110+50 to Sta. 111+50 on the uphill side of the road an old dry-masonry wall was replaced by a wet-masonry wall, which was given considerable batter

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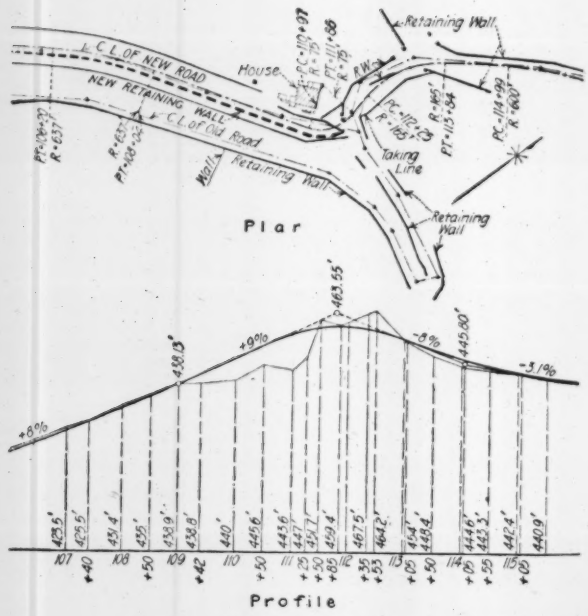


FIG. 2. PLAN OF RELOCATION OF NYACK-ROCKLAND LAKE ROAD AT STA. 112

on its front face in order to retain a good line of sight around the curve ahead.

The construction of the remaining portion of the road did not meet with any considerable difficulty. The typical construction is a 16-ft. macadam pavement on a 32-ft. roadway, narrowing to 24 ft. along the side-hill line. The foundation consists of an 8-in. course of field stone, laid on edge and filled with crusher run. Over the foundation is a 2-in. course of No. 4 (3½-in.) crushed stone filled with crusher dust. (Where unstable places occurred in the subgrade, the foundation course was increased to 12 in. in thickness and in several instances to 20 in.) Where the roadbed is in rock, the bottom course consists of a course of broken-stone macadam 4 in. thick. On the foundation a 3-in. bituminous top

FIG. 3. LOOKING NORTH TOWARD CURVE AT STA. 112 OF NYACK-ROCKLAND ROAD

FIG. 4. LOOKING NORTH THROUGH ROCK CUT AT STA. 112

course (penetration method) was placed, using a tar binder.

The binder selected for this road was "Tarvia X," spread at the rate of 2¼ gal. per sq.yd. in two pourings of 1½ and ¾ gal. per sq.yd. each. It was delivered to the road in motor trucks owned and operated by the Barrett Manufacturing Co. It was applied at a temperature of 300° F. under a pressure of from 40 to 60 lb. per sq.in.

The stone was obtained and crushed along the road by the contractor, using boulders from the hillside of talus, which was of an excellent quality trap rock. All structures were built according to the standards of the New York State Highway Department. Culverts were constructed of concrete with reinforced flat-slab tops. All earth ditches occurring on grades of 5% or over were paved with cobble gutters. On grades exceeding 7% the paving was laid in cement mortar.

The table shown on the next page gives the approximate quantities and unit prices of the contract.

B. H. Wait is division engineer for the State Highway Department. The contractors were Brady-Oltarsh Construction Co. of New York, William J. Kelly, superintendent. The writer had charge of the survey, preparing

of plans and supervising of construction. Livingston Leeds, assistant engineer, had immediate charge of construction.

COST OF NYACK-ROCKLAND LAKE ROAD

Clearing and grubbing	$200.00
12,500 cu.yd. earth excavation at	.40
4,800 cu.yd. rock excavation at	1.50
100 cu.yd. overhaul at	.01
12 tons of cast-iron pipe at	33.00
35 cu.yd. of riprap at	2.00
30 cu.yd. second-class concrete at	6.00
120 cu.yd. third-class concrete at	6.00
220 cu.yd. stone masonry at	4.00
1,500 sq.yd. cobble gutters at	.50
810 lb. metal reinforcement at	.05
116 lb. miscellaneous iron at	.05
2,600 lin.ft. wooden guard railing at	.23
9 guide signs at	2.00
2 highway number signs at	2.00
2 danger signs at	2.00
5 concrete sign posts at	10.00
120 cu.yd. broken stone, loose measure, No. 1 and No. 2 size at	1.50
3,800 cu.yd. foundation-course field or quarry stone at	1.45
1,775 cu.yd. bottom-course broken stone at	2.50
3,440 sq.yd. cleaning old pavement at	.004
2,050 cu.yd. top-course bituminous macadam, penetration method, at	2.75
1,932 lin.ft. restoring shoulders at	.02
1,250 cu.yd. stone masonry laid dry in place, complete, at	2.00
55,800 gal. bituminous binder at	.09
1,717 gal. bituminous binder, hot application at	.12
Approximate total cost of road	39,662.74
Approximate cost per mi.	13,480.10

Heat Loss Through Various Types of Window Sash*

In the industrial plant of today, where so much emphasis is placed upon maximum daylight, the exterior wall surface consists principally of windows, and the heat loss through them is a very important matter. A series of tests was made by W. S. Brown under the direction of Arthur N. Sheldon to determine the relative heat loss through various types of wood, steel and hollow-metal sash with single and double glazing.

The apparatus was an air-tight box one side of which was closed by the sash to be tested. An electric heater in the bottom of the test box maintained the desired temperature difference between inside and outside air. The loss of heat was computed from wattmeter readings, and a quantity *H*—the B.t.u. transmitted per 24 hr. per sq.ft. of exposed surface per degree difference in temperature—was determined for each sash. Tests were made at approximately 40° and 70° F. temperature difference under conditions closely imitating actual practice. Circulation of air over the surface of the sash to be tested was provided by two 12-in. fans placed one above the other 4 ft. from the vertical center-line of the sash. The humidity of the air inside the test box was kept as constant as possible throughout the series of tests by means of water-saturated felt and was measured by a wet- and dry-bulb hygrometer. The humidity remained constant during the period of each individual test.

A résumé of the comparative heat-transmission rate is given in Tables 1 and 2. The tests showed that for a given temperature difference the solid-steel sash bars were hotter than the wooden bars and by their greater conductivity constituted a direct path for the transmission of heat to the outside air. The hollow-metal sash bars were cooler than the solid-steel, but warmer than the wood.

Condensation appeared on the single-glazed sash when its temperature was low enough to cool below the dew point the air confined within the test box. The moisture was evidently on the inside of the panes. Where condensation occurred in the air space, in double-glazed sash, the moisture was always on the outside pane of glass, which of course, was the cooler one.

The wood sash showed no internal condensation, but in the two steel sash condensation was very marked and was evident in the hollow-metal sample. This condition was investigated in the steel sash, with a $\frac{3}{4}$-in. air space.

Holes were bored through each of several of the outer panes of wire glass, smoke was blown into the air spaces, and leaks to the outside or to adjacent air spaces were carefully noted. Leakage was found to be quite general. It was observed that in lights where condensation was evident the greatest leakage was into the interior of the box, and that in those lights where no condensation appeared the greatest leakage was to the air outside the box. Following this clue the entire sash was reglazed, and holes were bored in seven of the outer panes and in five of the inner panes, free communication being thus established either to the external cool air or to the warmer air inside. It was now observed that (a) lights in which the air space opened to the outside or cooler air only, showed no condensation; (b) lights in which the air space opened to the warm air inside the test box showed much condensation.

These experiments indicate that condensation in the air space of double-glazed sash can be eliminated almost entirely by connecting the air space directly to the outside air and at the same time effectively sealing it from the entrance of the warm air inside. The opening should be very small, equivalent to a $\frac{1}{8}$-in. hole, and should be protected from weather and dirt.

TABLE 1. HEAT-TRANSMISSION RATES AT APPROXIMATELY 40° F. TEMPERATURE DIFFERENCE

Sample	Actual Heat Head, Deg. F.	H Computed at Actual Heat Head	H Corrected to 40° F. Heat Head	Relative Humidity	Condensation
1. Single-glazed solid steel	44.5	29.3	28.5	36	None
2. Double-glazed solid steel—$\frac{1}{4}$-in. air space	37.4	20.4	20.5	34	None
3. Double-glazed solid steel—$\frac{3}{4}$-in. air space	45.7	22.8	22.5	43	Very slight on air-space side of one outside pane and box side of one inside pane
4. Double-glazed wood—$\frac{3}{4}$-in. air space	47.3	14.2	13.9	49	None
5. Single-glazed wood	41.2	25.6	25.5	44	Hardly noticeable
6. Double-glazed hollow metal—$\frac{3}{4}$-in. air space	43.9	15.4	15.2	54	On air-space side of one outside pane
7. Single-glazed hollow metal	46.0	27.1	26.4	43	All

TABLE 2. HEAT-TRANSMISSION RATES AT APPROXIMATELY 70° F. TEMPERATURE DIFFERENCE

Sample	Actual Heat Head, Deg. F.	H Computed at Actual Heat Head	H Corrected to 70° F. Heat Head	Relative Humidity	Condensation
1. Single-glazed solid steel	73.9	34.2	33.6	28	All
2. Double-glazed solid steel—$\frac{1}{4}$-in. air space	76.8	21.8	21.6	29	Fifteen outer panes out of 20 showed moisture on air-space side
3. Double-glazed solid steel—$\frac{3}{4}$-in. air space	76.0	24.2	24.0	30	One outer pane showed moisture on air-space side; one inner pane showed moisture on box side. Three lights showed both the above
4. *Double-glazed wood—$\frac{3}{4}$-in. air space	79.8	15.5	15.1	31	None
5. Single-glazed wood	67.6	28.9	29.3	29	Twelve lights
6. Double-glazed hollow metal—$\frac{3}{4}$-in. air space	75.6	16.6	16.4	36	On air-space side of one outside pane
7. Single-glazed hollow metal	77.7	30.5	29.7	25	All

*A test upon this same wood sash with the inside panes of single-thick plain glass instead of double-thick showed an increase in H of 3% at 70° F. temperature difference.

*Abstract of paper presented by Arthur N. Sheldon, Providence, R. I., at the annual meeting, December, 1916, of the American Society of Mechanical Engineers, in New York City.

Five Water-Works Make Filter Alum—I

SYNOPSIS—Columbus, Ohio, Trenton, N. J., Springfield, Mass., the Metropolitan Water District of the City of Omaha, and the Montreal Water and Power Co. are making their filter alum by treating bauxite with sulphuric acid. The cold process, producing crystallized alum in cakes, is employed under a patent granted to C. P. Hoover. After an introductory statement, each of the four plants in the United States is described by the engineer or chemist in charge. The Omaha description will appear in a subsequent issue.

Evolution and Status of the Process

The cost of chemicals used in water purification has become so high since the outbreak of the European War that anything new yielding equally good or better results at less expense is doubly welcome. Some 18 months before the war broke out, Charles P. Hoover, chemist in charge of the water softening and purification works of Columbus, Ohio, began a series of experiments on the manufacture of filter alum at the works, using low-grade hydrated aluminous material in its raw state. After two years of experimental and preparatory work he completed a plant and turned out the first product from it on Dec. 25, 1914. This plant was put in regular operation in January, 1915. The hot process was used and a syrup of alum produced, which after dilution was added to the water in the coagulating basin. Since then it has been found expedient to produce crystallized alum instead of using the syrup of alum. For reasons that will be made clear in the following paragraphs, the cold process instead of the boiling process of manufacture has been adopted.

Besides installations producing syrup of alum and also alum cake at Columbus, there are now plants for making alum cake by the later Hoover process at Trenton, N. J., Springfield, Mass., the works of the Metropolitan Water District of the City of Omaha, Neb., and the works of the Montreal Water and Power Co., supplying a portion of Montreal, Que., and various outlying districts. In addition to these five plants on public water-supplies there is one at the factory of the Independent Packers' Fertilizer Co., Columbus, Ohio.

Reduced to its simplest terms and assuming that the bauxite has already been pulverized, the Hoover process requires nothing but a small tank for mixing the baxuite and sulphuric acid, and a large shallow pan or box for allowing the reaction to take place, the water of crystallization to be driven off and the cake to harden so it can be removed. A small motor is required, together with weighing or measuring facilities and other accessories.

In the Hoover as compared with the ordinary commercial process, low-grade ore with relatively high iron content is perfectly satisfactory; and no refinement what-

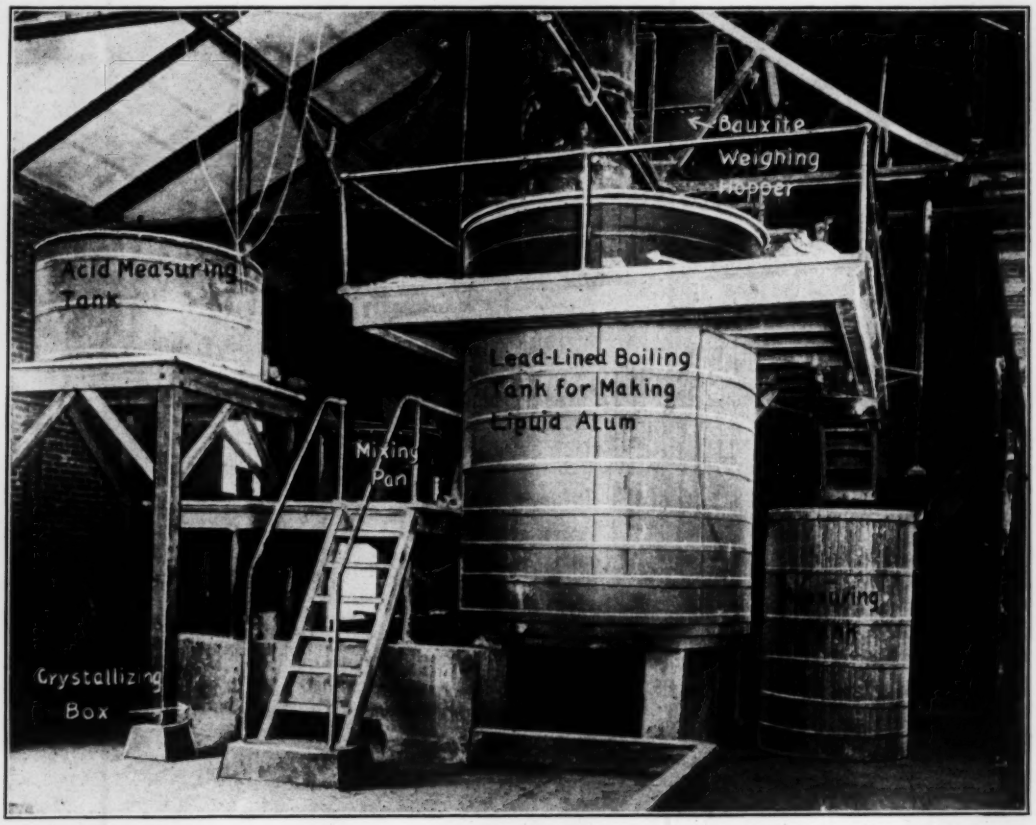


FIG. 1. OLD AND NEW PLANTS FOR MAKING FILTER ALUM, COLUMBUS WATER-WORKS

ever is attempted, since the foreign elements in the ore are believed to assist in the process of coagulation and precipitation. Moreover, the Hoover process requires less sulphuric acid than the ordinary commercial process and, what is of great importance in the operation of a water-coagulation plant, produces a more highly basic sulphate of alumina.

Under date of Sept. 5, 1916, there was issued to Mr. Hoover, United States Patent No. 1,197,123, containing claims for not only the manufacture of sulphate of alumina by the process already outlined, but also for the treatment of water by the process in coagulation and sedimentation basins. The Hoover Chemical Co. has been organized to promote the process in the United States, with George A. Johnson, consulting engineer, 150 Nassau St., New York City, as manager of the company. The Hoover Chemical Co. for Canada has also been organized. Its manager is J. O. Meadows, 20 Charlevoix St., Montreal.

Mr. Meadows, it may be added, is engineer in charge of the operation of the water-purification plant of the Montreal Water and Power Co., already mentioned. The Columbus and Omaha plants were designed by Mr. Hoover, the Trenton and Springfield plant by Mr. Johnson, and the Montreal plant by Mr. Meadows.

It is well to state in passing that bauxite is found in various parts of the country and can be bought from a number of different companies. Since, as already stated, low-grade bauxite is employed, the price of the ore at the mine is very low—lower than the freight rates when considerable distances for transportation are involved. Halloysite or other native hydrated aluminous materials might also be used, but thus far the bauxite has proved to be more tractable than halloysite.

Mr. Hoover's description of his process, as given in his patent already mentioned, makes very interesting reading to those deeply concerned with the subject; but as it is lengthy and can be obtained from the Patent Office for a nickel, direct citations from it will not be made here.

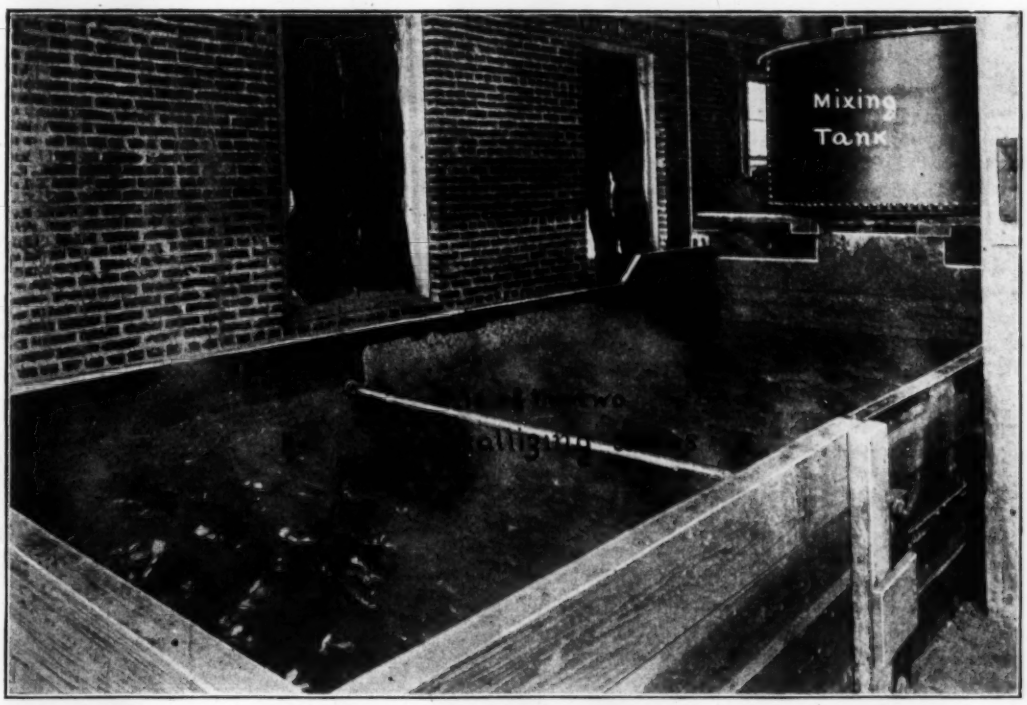


FIG. 2. MIXING TANK AND ONE OF TWO CRYSTALLIZING BOXES, TRENTON ALUM-MAKING PLANT
The mix is boiling by its own heat and giving off vapor of crystallization

Descriptions of the four plants on municipal waterworks in the United States will now be given. For reasons that need not be gone into here, the Montreal plant cannot be described at the present time.

Old and New Plants at Columbus, Ohio

By Charles P. Hoover*

The original, or syrup-of-alum, plant at Columbus was the first one to be built. This differs from all the other plants in that the alum is not crystallized. The bauxite, sulphuric acid and water are boiled together in lead-lined tanks. As soon as the solution becomes basic, which usually requires boiling with steam for from 6 to 8 hr., the resultant alum liquor is diluted and then fed into the water under treatment. In the new, or cold-process, plant no steam is required, the heat resulting from the reaction of the acid and the bauxite being sufficient to produce a basic alum. Both the old and the new plants are shown by Fig. 1.

In the new plant the bauxite is weighed in a hopper and the acid in a box, each a part of the old, or liquid-alum, plant. The weighed quantities of acid and bauxite are discharged into a mixing pan, elevated 7 ft. above the floor. This pan is 4 ft. in diameter, 2½ ft. deep and is provided with a double-motion agitator. Bauxite to

*Chemist in Charge of Water Softening and Purification Works, Columbus, Ohio.

the amount of 1,000 lb. and 500 lb. of sulphuric acid are placed in the mixing pan and mixed for a period of 5 min. Then the mixture is discharged into the crystallizing box below, where it spreads out on the floor of the box in a thin layer. In another 5 min. another batch is discharged on top of the first, and this is continued until seven or eight batches have been deposited. In 10 or 15 min. after the mixture is discharged into the crystallizing box, it begins to boil violently, rising to a height of 12 or 14 in. On cooling, the mixture crystallizes into an alum cake having a thickness of from 3 to 3½ in.

The alum is usually made one day and removed the following day. The capacity of a plant depends entirely on the space provided for crystallization. At Columbus 484 sq.ft. are provided, and 5 tons of alum are produced in a day. The time required for mixing the bauxite and the acid is only 35 min. for the seven batches, and it usually requires two men about 3 hr. to remove the 5 tons of alum from the crystallizing pan and pile it in the storage house. The pan is 22x22 ft. in plan and 20 in. deep. It was formed by building a concrete wall on an existing concrete floor.

Low-Cost Plant at Trenton, N. J.

By F. W. Daggett*

Trenton, N. J., since May 1, 1916, has been making the alum used as a coagulant in its mechanical water-filtration plant. The filter plant has a nominal daily capacity of 30,000,000 gal. and is now being operated at an average rate of about 16,000,000 gal. per day.

The bauxite from which the alum or sulphate of alumina is made is mined in Arkansas, ground and put up in 175-lb. bags in East St. Louis, Ill., and shipped from there to Trenton, where it is delivered by the Aluminum Co. of America for $20 a gross ton, f.o.b. at the filtration plant. The ground bauxite has the appearance of cement, with a reddish-brown tint, but is not ground so fine as cement. It contains about 55% of alumina. The sulphuric acid, which is the only other material used in the manufacture of the filter alum, comes in 55-gal. drums. A 60° Bé. commercial acid is used, which can be bought at present for about $20 to $25 a net ton.

The bauxite and sulphuric acid are conveyed by elevator to the fourth floor of the filter building. The bauxite is emptied from bags through wooden chutes into the mixing tank. The acid is siphoned out of the drums. The siphon consists of (1) a piece of steel pipe extending within an inch of the bottom of the drum with its upper end screwed into a fitting in the bung hole of the drum, (2) a short horizontal line of steel pipe connected with the vertical line by a hard-rubber union and (3) a length of rubber hose especially made with a minimum amount of fiber. The pipe is greased for protection against the acid. To start siphonic action, a hand air pump is used, its rubber tube being connected with the fitting in the top of the drum.

The mixing tank is of steel, 6 ft. in diameter and 3½ ft. high (see extreme right of Fig. 2). The tank is not lined. The acid and bauxite are stirred by a revolving paddle having cast-iron teeth. The stirring or mixing process requires 3 to 5 min. It is necessary to discharge the mixture immediately after it is ready and before reaction begins to take place. The mixing-plant tank is emptied by removing one or the other of two rubber stoppers, one placed on each side of the tank, directly over one of the crystallizing boxes.

There are two crystallizing boxes, each about 8x18 ft. in plan with an available depth of 2 ft. (one of these is shown in Fig. 2). These boxes are made of 3-in. yellow pine and have their bottoms covered with paving brick.

When the mix is discharged into the tanks, which are used on alternate days, it looks like liquid paint. In the process of crystallization, the required heat is supplied by the reaction between the acid and the bauxite, so that no extra heat is required. In about 10 min. the mixture comes to a boil (Fig. 2) and gives off steam. In about a half-hour the major reaction is complete. Some 24 hr. is allowed for the hardening and cooling process, although

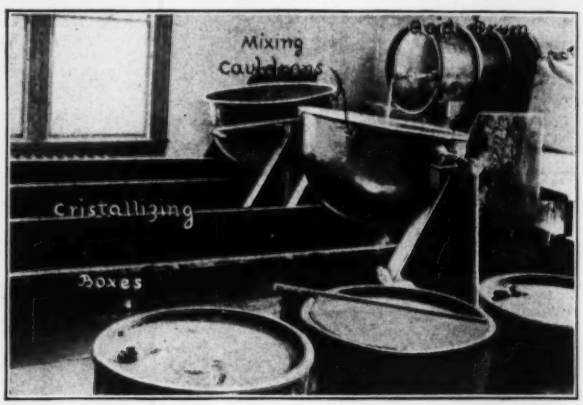


FIG. 3. MIXING CALDRONS AND ALUM CRYSTALLIZING BOXES, SPRINGFIELD, MASS.

perhaps the product could be removed sooner. It suits local conditions at Trenton to remove the alum from the crystallizing box at night, making use of the night shift, which has plenty of time for the purpose.

The layer of alum, or sulphate of alumina, formed in the plant as it is now being operated at Trenton, is some 3 in. thick. It is broken up by a Crown No. 56 coal pick or pneumatic hammer made by the Ingersoll-Rand Co. This tool the city had on hand, having bought it originally for cutting sheet asphalt. A pneumatic railway-tie tamper with a cutting edge has been considered; but the coal pick, as has been said, was on hand, and it works well. Prior to the trial of this tool, hand labor was employed, making use of chisels and picks. These did the work, but not so readily as the pneumatic hammer.

A 6x6-in. Ingersoll-Rand vertical compressor with a capacity of about 60 ft. per min. under 80 lb. pressure, driven by a 10-hp. d.c. Westinghouse motor, operates the air hammer and paddle in the mixing tank—each is required, but not at the same time.

The plant described produces about 2,500 lb. of filter alum per day. It is said that double that amount could be produced by the same installation, if it were needed. The total cost of the plant was about $2,200. Some of the material was on hand, including the tanks, but the latter had to be altered. As the plant stands, the tanks, crystallizing boxes, etc., cost about $600; the air compressor, $350; and the motor, $150.

The cost of the filter alum produced by the plant is about $32 per ton, but at current prices for acid it would

*Superintendent of Water-Filtration Plant, Trenton, N. J.

not be over $25. Commercial filter alum is now selling for something like $60 per ton. The city is paying Mr. Hoover a small royalty. According to the present outlook the city will be able to make a net saving, for the first year of its alum manufacturing, of about $6,000.

The Trenton filtration plant is operated under the direction of the writer, who was engineer on the construction of the plant in behalf of Johnson & Fuller, consulting engineers of New York City. Alvin Bugbee is superintendent of the Trenton water-works.

Simple Plant at Springfield, Mass.

By Elbert E. Lochridge*

On July 5, 1916, a plant for the manufacture of sulphate of alumina at the West Parish filters, Springfield, Mass., was put into operation. As the method of applying the sulphate at this plant is that of overdosing and intermittent application, the total amounts required are less than would normally be provided for a plant delivering approximately 12,000,000 gal. of water per day.

The normal capacity is 1,000 lb. of alum per day; and it is actually operated substantially every day at this rate, although amounts up to 1,200 lb. have been delivered on certain days.

The plant (Fig. 3) consists of two 83-gal. iron caldrons, suspended above the end of two wooden crystallizing boxes, each 8 ft. in length, 3 ft. in width and 2 ft. deep. The sulphuric acid and bauxite are mixed in these caldrons and dumped directly to the crystallizing boxes, where the chemical reaction takes place. A steel plate is fitted in the bottom of these boxes to facilitate breaking the cake, and the wood is treated with acid-proof paint. Just at the completion of the reaction and before the cake has become too hard, it is cut up into small squares by pushing a crowbar along the plate, and it hardens in this form.

As the method of application to the water at this plant is "dry feed," a 15x8-in. Type D Jeffrey crusher with a 10-hp. motor is used to crush the alum to proper size for use. The plant is situated on the second floor of the main building, and from this second floor the finished product is fed directly to the hopper above the screw conveyor delivering the alum to the water.

The bauxite and acid now on hand cost respectively $17.50 and $65 per ton, f.o.b. Springfield, Mass. In the case of the bauxite this included shipment from Arkansas to East St. Louis, pulverizing at that point and reshipment to the plant. The cost per ton at East St. Louis is $12. The acid cost includes 135 drums and shipment from Chicago. Very much better quotations have been received since these amounts were paid. With the crusher now installed it is planned to pulverize the bauxite at the plant, thus saving the shipment via East St. Louis. This will reduce the bauxite cost at least 50%.

It is rather difficult to figure labor cost with the short period of operation, as the work is carried on by our regular men and no extra labor is employed. The cost of hauling all material from the railroad station to the filtration plant, 5 mi., is $1.75 per ton. The total cost of the plant, in addition to the royalty to the Hoover Chemical Co., was $756. This includes crystallizing boxes, crusher, motor, caldrons and labor.

*Chief Engineer of Water Board, Springfield, Mass.

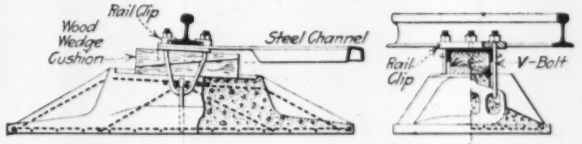
The plant has been in operation constantly since it was started and has never failed to produce sulphate of alumina of a satisfactory quality. Comparative tests on its coagulation properties per pound have been carried on to determine proper dosing, and in this manner the strength has been kept substantially the same as that of regular market alum as formerly used.

Concrete Ties in Holland

An experimental track in which the ties are composed of pairs of concrete blocks connected by transverse steel tie-bars is in service on the Amsterdam-Utrecht line of the Netherlands State Railways, in Holland. In principle the tie resembles the Kimball ties tried in this country on the Chicago & Alton Ry., and the Denham-Olpherts cast-iron plate ties that have been used extensively in India.

Each tie is composed of a pair of reinforced-concrete blocks about 4½x3 ft. on the base, with a maximum height of about 12 in. The top surface is about 36x18 in. with flaring ribs to the corners of the base, and its top forms a seat for a pair of wedge-shaped wood blocks. Upon the wood cushions of a pair of concrete blocks rests the ends of a transverse inverted steel channel. Upon this rest the 92-lb. T-rails and rail clamps.

The rail fastenings are of curious design. At each block the rail is held by a pair of deep V-bolts, or stirrups, the loops of which pass through the projecting ends of cast-iron anchors embedded in the concrete. The threaded ends of the bolts pass through the rail clamps. In surfacing track the adjustments are made by loosening these bolts and adjusting the wood wedges, the concrete blocks being left undisturbed, as tamping is liable to result in causing cracks.



TRACK WITH CONCRETE BLOCKS AND TRANSVERSE STEEL CHANNELS; NETHERLANDS STATE RAILWAYS

The weight is about 440 lb. for each block. The weight per lineal foot of track is about 1,500 lb. (including ballast filled over the base of the blocks), and the cost about $6.20, as compared with 600 lb. and $5.75 for the ordinary track construction. But this weight and cost for the experimental track are obtained at the expense of the support of the rail, the spacing being 4 ft. c. to c. (or 2 ft. at the rail joints). Such wide spacing of rail supports is undesirable in track construction, and is only introduced to minimize the cost. The experimental track is only 120 ft. long. It carries considerable traffic, but the trains run at low speed, as it is near the city of Utrecht. It has been in service for about two years and is said to be generally in good condition, although cracks have developed in some of the blocks.

Dredge Raises Fuel Barge—The combination suction and dipper dredge "Monticello," belonging to the Government, raised a barge loaded with sawdust near St. Helens, Ore., in the latter part of November. The load of sawdust was transferred by bucket conveyor from the sunken barge to another moored alongside.

Roosevelt Drainage Tunnel of the Cripple Creek District

SYNOPSIS—The Roosevelt drainage tunnel is the most important mine-drainage project since the famous Sutro tunnel of the Comstock mines, Nevada. It is an undertaking that will cost nearly $1,000,000. A historical sketch of the drainage of the district is given.

The Cripple Creek mining district is an area of eruptive or volcanic rock, of an irregular oval shape, 3 to 5 mi. across and of unknown but probably great depth. Small as this district is in area, it has already produced $360,000,000 in gold (since its discovery in 1891) and it is still producing at the rate of $15,000,000 yearly.

The district is entirely surrounded by impervious granite walls. It is apparently an immense crater. This volcanic mass has been fissured by many successive eruptions and contains innumerable cavities and crevices of all sizes and shapes. During all the ages that have elapsed since the eruption, surface water has permeated down and through these cavities until the crater was saturated up to the notches in the edges of the granite walls. T. R. Countryman, who has studied the district for many years, estimates the storage of this underground reservoir to be at least 40,000,000 gal. per vertical foot.

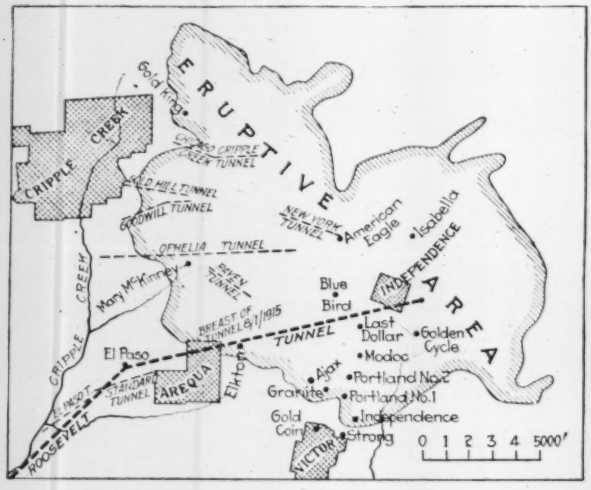


FIG. 1. MAP OF CRIPPLE CREEK MINING DISTRICT AND DRAINAGE TUNNEL

Early Problems of Drainage

In the earliest days of Cripple Creek mining the water question was not a serious one; but as the important mine shafts increased in depth, the seepage began to be very troublesome, and below 500 ft. the mining operations were greatly hampered. The original water level was about 9,500 ft. above sea level on the western side of the district and 9,700 ft. on the eastern side (about 200 ft. below the surface). The first mines to do considerable pumping were those in the southwestern corner of the district, notably the El Paso and the Elkton.

At first, each mine constructed its own pumping plant; but as the depth of the workings increased, the cost of pumping became enormous. Several joint pumping plans were tried, but none was very successful. The pumping plant at the Portland mine, installed in 1898, cost $3,000 a month to operate and handled only 525 gal. per min.

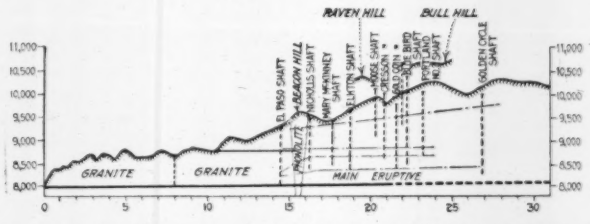


FIG. 2. PROFILE OF ROOSEVELT DRAINAGE TUNNEL, CRIPPLE CREEK, COLO.

The cost of the plant was not less than $50,000. The Elkton mine in an effort to develop the 800-ft. level pumped continuously from 1901 to 1902, but had eventually to give up.

About this time the first tunnel of the district, constructed solely for drainage purposes, was undertaken. This was the El Paso tunnel, constructed by the El Paso mine to drain its lower workings. Previous to that there had been three tunnels that had served for drainage as well as for workings, the most important of these being the Standard tunnel. The Standard tunnel was driven in 1895, and it is estimated that it discharged 12,000 million gal. by 1901, when it became dry. The El Paso

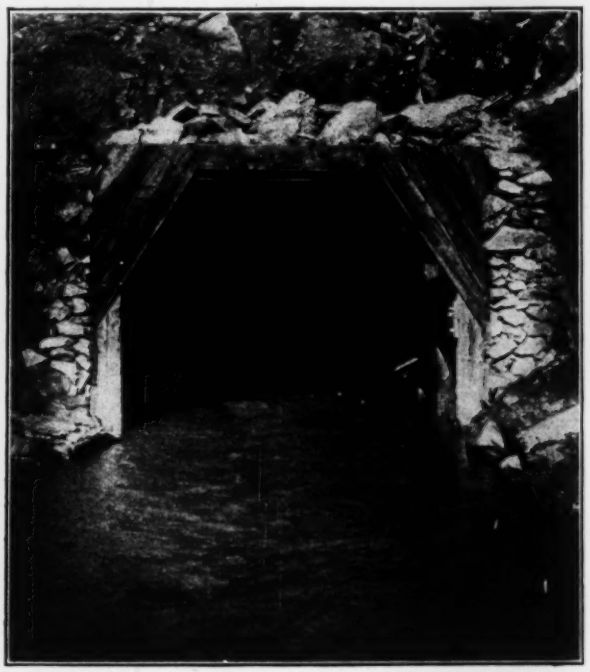


FIG. 3. PORTAL OF ROOSEVELT DRAINAGE TUNNEL DISCHARGING 17,000 GAL PER MIN.

tunnel was begun in 1902. It is 5,000 ft. long and of a cross-section 4½x7 ft. It cost $90,000, and it is estimated to have saved $1,150,000 in pumping. It lowered the water in the Elkton mine shaft more than 5,000 ft. distant, nearly 200 ft. The elevation of the El Paso tunnel portal is 8,786 ft. and it was driven at a grade of 0.28 ft. per 100 ft., or 1 in 357.

When the El Paso tunnel had served its purpose, a larger and deeper tunnel was planned. Every hundred feet added to the depth of a drainage tunnel in the district meant 1,000 ft. added to the length and $30,000 to the cost of the work. No particular engineering difficulties were anticipated or have developed, but the work has proved a test of strength and endurance.

Beginning of the Roosevelt Tunnel

The elevation of the floor of the Roosevelt tunnel at the portal is 8,034 ft. above sea level (making it 750 ft. lower than the El Paso tunnel), and it has a grade of 0.3 ft. per 100 ft., or 15.9 ft. per mi. (1 in 333). Its depth below the surface at the El Paso mine shaft is

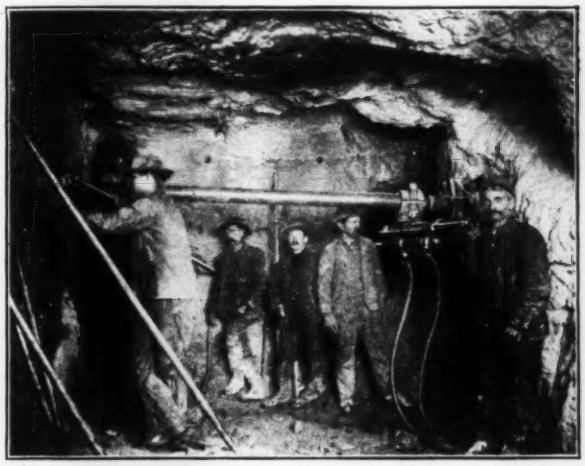


FIG. 4. HEADING IN ROOSEVELT DRAINAGE TUNNEL

1,289 ft., and at the Elkton mine shaft is at a depth of about 1,640 ft. It discharges into Cripple Creek Cañon about 5 mi. below the City of Cripple Creek.

The tunnel cross-section as originally designed was 10 ft. high by 7 ft. wide in the clear, with a waterway 3 ft. deep by 6 ft. wide on one side and a shelf on the other for the mine-car tracks.

The work was begun on June 1 and continued until Dec. 1, 1907, when the original contractor abandoned the contract. Work was continued for 300 ft. and again abandoned until January, 1908, when a contract was let to A. E. Carlton, a well-known Cripple Creek banker and mine owner. The cross-section of the tunnel was then changed to 10 ft. wide by 6 ft. high, with a ditch or waterway on one side 5x3 ft. in cross-section.

At a distance of 8,000 ft. from the portal an intermediate construction shaft was sunk 700 ft. deep to the tunnel level, on the tunnel center line. From the bottom of this shaft the work was started in both directions and thereafter was pushed at all three headings night and day.

All the tunneling as far as the El Paso mine shaft, 14,500 ft., was in hard granite, in which it required 24

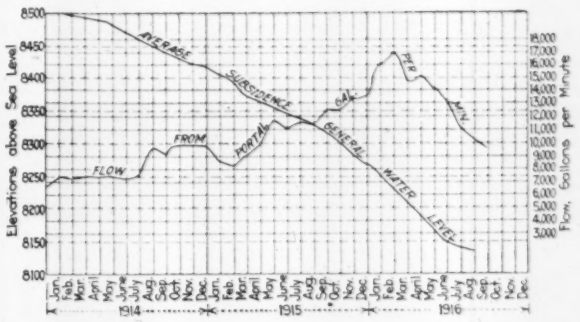


FIG. 5. EFFECT OF ROOSEVELT DRAINAGE TUNNEL ON MINE WATER LEVELS

to 30 holes and sometimes 300 lb. of powder to break a round 5 ft. deep. The best monthly record was 400 lin.ft. in October, 1908. The cost of this part of the tunnel was $27.27 per lin.ft. The center line of the three headings differed less than 2 in., while the grade of the floor was practically correct. The tunnel reached a point opposite the El Paso mine shaft on Aug. 1, 1910. Here a deflection to the right was made, as the plan, Fig. 1, shows.

By March, 1911, the funds of the tunnel company were practically exhausted, and the tunnel was stopped about 1,000 ft. beyond the El Paso mine shaft. In the fall of the same year the company raised additional funds, and the work was resumed in the main heading to a total distance of 16,857 ft. from the portal. At this point a large open fissure, or vein, accompanied by a heavy flow of water, was encountered, together with caving ground, which made the work extremely difficult, and it was abandoned in March, 1912.

In June, 1914, work was again resumed with the intention of driving the tunnel on to the Vindicator mine, a distance of 10,000 ft. When the work on the original tunnel stopped in March, 1912, all pipe, track, ven-

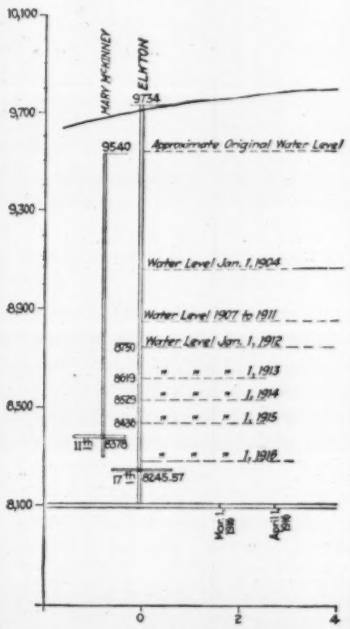


FIG. 6. EFFECT OF ROOSEVELT TUNNEL AT ELKTON MINE

tilating apparatus, etc., had been taken out. All these had to be replaced, and 1,100 ft. of ditch had to be cut in the tunnel already constructed.

Drilling was commenced in the heading on Aug. 3, 1914, and it was deemed advisable to make a detour to the north of the old tunnel line to avoid the bad cave in the breast of the original tunnel. This detour was commenced 55 ft. back from the breast and ran in a northeasterly direction for a short distance and then back to the line of the original tunnel. Some bad ground was encountered soon after the commencement of the work, and considerable heavy timbering had to be done. By September the bad ground had been passed, and good progress was made until Nov. 4, 1914, when the El Paso mine plant was destroyed by fire, and work on the tunnel had to cease.

The equipment for driving the tunnel consisted of a track of 20-lb. rails and a motor and blower in the tunnel a short distance from the El Paso shaft on the portal side, where there is good natural ventilation. The ventilation in the breast is supplied by a 16-in. pipe. The compressed air for the drills was obtained from the El Paso compressors and was taken to the breast in a 2½-in. pipe. Stables for three mules and a machine shop for minor repairs were situated near the blower. The tunnel was lighted by electricity.

Work was begun again early in 1915 and has been continued to date. In the spring of this year, after a connection with the Elkton mine shaft, the ventilating machinery, mule stable, etc., were moved to a new location near the foot of this shaft. The tunnel is now over 4 mi. long and over 1,800 ft. below the surface. Its width is 9 ft., and it is 8 ft. high in the clear, with a ditch or waterway on one side 4x2 ft. deep. The tunnel has passed out of the granite and is now being driven in a hard volcanic breccia with occasional dikes of basalt and phonolite, which are both very hard.

Usually it requires 30 holes to break a round only 5 to 7 ft. deep. Very little timbering, however, is necessary. The work is being done with three shifts of eight men, at a cost of from $20 to $25 per lin.ft. The rock is hauled from the breast by a mule and is now taken out of the Elkton mine shaft. Previous to the connection with this shaft, all the mucking had to be done via the El Paso shaft, 1 mi. distant. The mucking cars had to travel in water 2 to 3 ft. deep. The drills used are Ingersoll-Leyner Model 18, with 2½-in. bits for starting. Two drills on a horizontal bar are worked in each shift. The best record yet made is 456 lin.ft. in September of this year.

Aside from the point where the tunnel was abandoned on account of caving ground, the most inconvenience has been caused by the water, which flows in from all directions, sometimes in streams of 5,000 gal. per min. These often spurt out of the heading into the faces of the workmen, who require patience and persistence to keep at work. The connection with the Elkton mine shaft was a particular instance, as the junction was driven upward from the tunnel cross-cut, and great quantities of water were continually coming down upon the heads and shoulders of the drillers. Fortunately the water is at a temperature of between 60° and 70° F. and is not so disagreeable to work in as it would be were it cold or hot.

The tunnel has discharged as much as 17,000 gal. per min., as the diagram in Fig. 5 shows. The flow, however, is now rapidly falling off, which substantiates the theory that the reservoir is being rapidly drawn down. The diagram also shows the effect of the tunnel on the water levels at the Elkton mine shaft and in the mine shafts of the district as a whole.

T. R. Countryman, consulting engineer, Cripple Creek, Colo., is chief engineer of the tunnel project and has been associated with it and with previous tunnels since their inception. He is indeed the original projector of the scheme of draining the district by tunnels. The work in 1914 was supervised by E. P. Arthur, Jr., as engineer. Since then the work has been supervised by Charles F. Fuller as superintendent. The facts given are based on a trip through the tunnel by one of the editors and on material kindly furnished by Mr. Countryman.

Dispatcher's Selective-Control Railway Signal System

By John B. Harlow*

By combining a block-signal system with equipment developed for telephone train dispatching, the Western Electric Co. has evolved a unique railway communication system that enables a train dispatcher to set signals anywhere on his division, to know when the signal moves and from what direction a train enters a block—all without the aid of the way-station operator.

This system was shown at the recent annual convention of the American Electric Railway Association in Atlantic City, but it is of as great interest to steam roads as electric interurban lines.

The basis of this system really is the new so-called alternating-current selector developed for telephone train dispatching. In the regular train-dispatching circuit there is a selector located at each way-station, while in the dispatcher's office there is a bank of keys, one for each way-station selector. With that equipment in place, when the dispatcher wishes to call any particular station, he operates the key marked for that station. The key sends impulses over the line in a predetermined combination that will close the contact of the selector at that particular station and operate a bell to call the way-station operator.

This new selector, shown in Fig. 1, operates on reversals of polarity at a frequency of about 3½ cycles per second. Since it is operated through a condenser, it will not respond to uni-directional impulses and is therefore particularly well adapted for use on lines that might be crossed with telegraph, trolley or other wires carrying direct current. The impedance of the selector to alternating currents of commercial frequencies is so high that it would take an extremely high voltage across the selector terminals to operate it at all. Thus, the chance of false closure of a contact due to a foreign source of potential is reduced to a minimum.

In the control of signals the selector contact is used to operate a relay in place of the bell. For this service the selector is provided with two contact points, one of which is used to set the signal and the other to clear it, as shown in Fig. 2. When the set contact is closed, the setting relay is energized and will open the feed to the

*Engineering Department, Western Electric Co., 463 West St., New York City.

line relay controlling the signal, causing the signal to come to the stop position. The setting relay locks up through the back contact of the clearing relay. Thus, the feed to the line relay of the signal circuit is held open until the dispatcher closes the circuit that energizes the clearing relay.

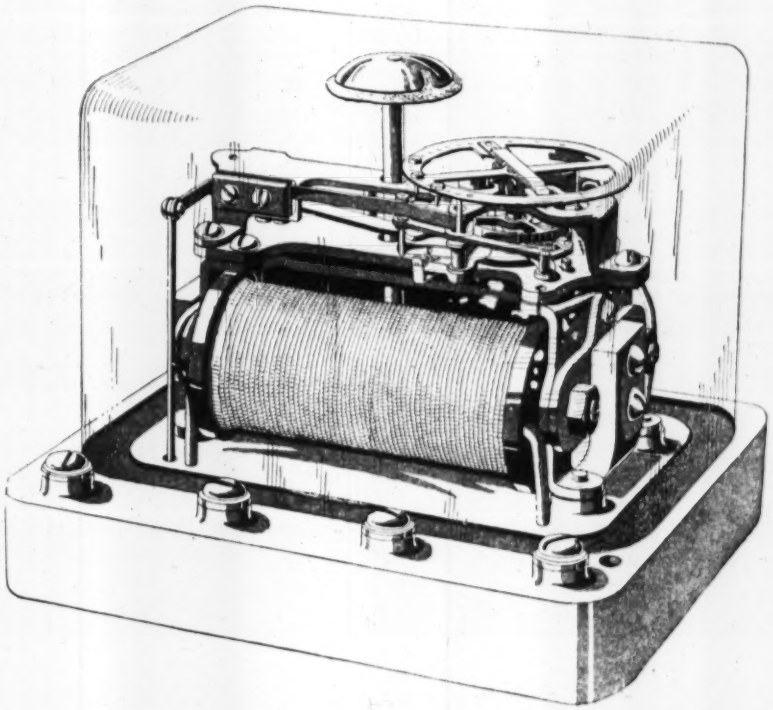
Since a scheme of this sort to be of any great use to the dispatcher must give some indication that the signal has responded correctly, a simple but effective form of answer-back or indication is used. In series with the setting relay is a second relay that controls the answer-back. This relay on being energized starts up a small motor that revolves a shaft at about 40 r.p.m., on which are mounted toothed wheels. These teeth are cut so that a definite code will be given in the same manner as in a fire-alarm or messenger-call circuit.

Each signal will have two distinct codes, one for its set position and one for its clear position. This is accomplished by having on the motor shaft two wheels which are connected in the answer-back circuit through contacts on the line relay, the particular wheel being determined by the position of the line relay. This code is repeated as long as the dispatcher holds the contact closed on the selector.

These signals can be used strictly as train-order signals entirely separate from the automatic block circuit, or they can be the home signals of the automatic block system itself. Fig. 2 shows the application to the home signals of an automatic block circuit. The system in no way interferes with the safety of the block-signal circuit, as it is impossible for the dispatcher to clear a signal if a train or car occupies the block. Thus, all that can happen is that a signal will be held at the stop position.

In the Western Electric exhibit at Atlantic City a working model of this system was shown. The block-signal circuit used was the traffic-direction block, or "T.D.B." system of the Union Switch and Signal Co. This system has relays in its circuit, which can readily be used to give the dispatcher information regarding the movement of trains in a given block by the operation of a selector, either separate from one of the signal selectors or by utilizing a third point on a signal selector. A relay is closed, which starts up the answer-back or code-sending motor; and one of three different codes will be sent to the dispatcher as long as he holds the contact of the selector closed. If no train is in the block, one code will be given him; if a train has entered the block from the east, a second code will be sent; and if a train has entered the block from the west, a third code will be sent. The way this is done is shown in Fig. 3.

The signal circuit makes use of a center-fed track circuit, and located in the circuit are two stick relays S1 and S2. The circuit is so arranged that a train entering from the east, track relay No. 2 being deënergized, will energize stick relay No. 2, which will stay locked up until the train has passed through the block or has left it either by backing out or going into a siding. A train entering the block from the west, track relay No. 1 being deënergized, will energize stick relay No. 1, which will come up and lock up until the train has left the block. By means of contacts on these relays, as shown in Fig. 3, it can be seen that one of three different code wheels can be connected to the answer-back circuit, (1) when both stick relays are deënergized, (2) when stick relay No. 1 is energized, or (3) when stick relay No. 2 is energized.



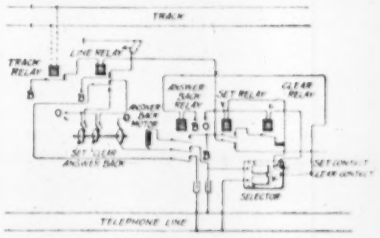


FIG. 2

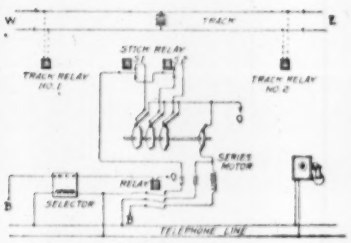


FIG. 3

SELECTOR AND CIRCUITS FOR NEW DISPATCHER'S TRAIN-SIGNAL SYSTEM

By having this apparatus in each block it is possible for the dispatcher to inform himself of the state of traffic at any point.

It must be remembered that this system is used merely as a means of conveying information to the train crews or to the dispatcher. It in no way affects the safety of the signal system and is merely superimposed on the signal system to enable the dispatcher to keep in close touch with his trains without the aid of a third party—the way-station operator. In this way the system looks toward the final elimination of the operator at the way-station.

The Miami Valley Flood-Protection Work

I—Fixing Maximum Flood Limits

A Pioneer Study of Storm Records in the United States; Analysis of Storm Frequencies, and Prediction of Future Storm Possibilities in the Central Region; The Final Result, Ten Inches Runoff in Three Days

After the terribly destructive flood of Mar. 23-28, 1913, the people in the valley of the Great Miami River, Ohio, made an urgent appeal for protection against future floods. This put a wholly new problem before the engineers who were called in to devise such protection. They had to determine what future great floods might visit the valley; or, still more difficult, what is the *greatest* flood that may come. No such forecasting had ever been done for a stream and a drainage area like the Miami, or for intense rainstorms covering thousands of square miles.

While this problem may seem like one of pure prophecy, it was actually solved by means of a monumental research into rainfalls and floods. Records were fully analyzed, and their facts extended into the future by an interesting combination of judgment and calculation. From an examination of this work, it can be asserted with all confidence that the first premise of the great Miami flood-protection enterprise rests on a thoroughly sound basis.

The research is of entirely new nature. Similar work will undoubtedly be needed elsewhere in the future. Fortunately, many of the data collected in the present research are applicable to other regions; but some of them are valid for the Central States region only, and possibly are even more limited than this. The distinction between the two classes of results must be carefully observed.

The Starting-Point of the Work

To state the final results in advance, Figs. 1 and 2 are given. The former shows the Dayton plain in flood in 1913, and as it would be flooded by the ultimate maximum flood. It also shows how both floods are to be controlled by the retarding basins, which form the central feature of the flood-protection works soon to be built.

Fig. 4 gives the adopted flood-flow curve, the runoff of the ultimate greatest storm covering the entire valley, as used in designing the Miami plan. Comparison of the two upper lines of the intensity diagram with the dotted line, showing the 1913 intensity, expresses clearly what excess is provided for. The same appears from the diagram of accumulated flow, the right-hand diagram in Fig. 2. For better understanding of the two diagrams, the rate of flow is expressed not only in inches depth, but also in second-feet per square mile. In the accumulated-flow diagram the retarding-basin capacities (to spillway level) are represented, showing how little is left for the stream channels below, even in the greatest flood.

Though the 1913 flood, which overtopped the levees at Hamilton and Dayton by ten or a dozen feet, was greater than any Miami flood in living memory, yet it was taken as a storm that undoubtedly must be protected against. Further, the engineers brought with them knowledge of some of the greatest storms recorded in the Central States. The largest of these, the storm of October, 1910, centering near the mouth of the Ohio River, in Illinois, Kentucky, Tennessee and Missouri, was typical of summer or fall storms in having a low ratio of runoff; yet in total amount of rainfall it was about 45% greater than that of 1913 for an area equal to the Miami watershed. It was kept in mind as a provisional maximum, while detail investigations were being made.

There was no lack of popular belief that such a flood as the one of 1913 could not happen again, that it is not in the regular order of natural phenomena, but represents a cataclysmic happening. At the same time it developed that even men in close touch with weather and flood matters were not fully informed as to great storms. Under the circumstances the engineers felt it vitally necessary to look up storm records in the most thorough manner possible, with a view to supporting or modifying their views by the results of study of past

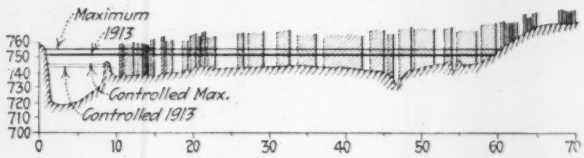


FIG. 1. CROSS-SECTION OF THE DAYTON PLAIN
Flood of 1913, with actual possible and controlled flood stages

flood history and estimate of future possibilities. Such a research was at once entered upon.

Confirmation of the starting-point ideas was obtained in a very short time. On bringing together the historical records of the Miami Valley and reducing the vague data of 70 and 100 years ago to figures as well as trained judgment would permit, two facts came out: First, since the coming of the white men there has been no Miami flood equal to that of 1913, not merely in gage-height and extent of inundation, but also in volume. Second, indications are that one or two prior floods came within reaching distance of 1913. The 1913 flood, while considerably greater than earlier recorded ones, is not so pre-eminently ahead of them as to constitute a cataclysmic phenomenon. Rather, it forms a natural and harmonious element of a progressive list, though for the Miami it indisputably stands at the top of the list. This latter fact was modified and made very prominent by the subsequent researches, which examined conditions outside the Miami Valley.

Great Storms and Floods in the Past

Two points had to be determined by the research: What storms and floods have happened in the past, and what are to be expected in the future. No physical evidences that gave help in the matter were found in the valley. The limited period of the records for the Miami needed extending by the aid of records from other regions.

Studying the records available at Dayton and in the Columbus office of the Weather Bureau proved insufficient.

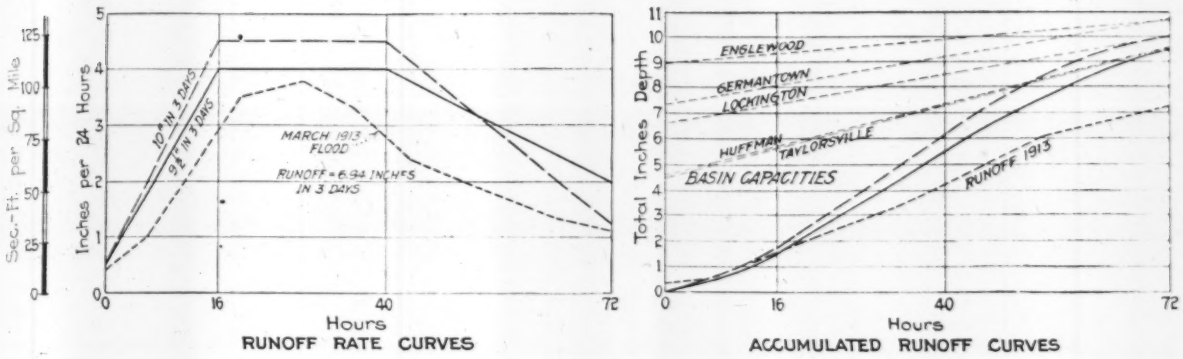


FIG. 2. RUNOFF AND ACCUMULATED RUNOFF CURVES FOR THE MIAMI VALLEY; THE MAXIMUM POSSIBLE, AND THE 1913 FLOOD

The capacities of the five retarding basins are represented by the straight lines in the right-hand diagram; the ordinate at zero hours represents the initial capacity, while the other ordinates represent initial capacity plus conduit discharge.

Therefore several men were sent to Washington and spent eight months there, abstracting the entire storm rainfall records of the country.

The Miami area, 120 mi. long, northeast to southwest, by 50 mi. wide, containing 4,000 sq.mi. (exclusive of the tributary Whitewater near the mouth), lies in 40° to 41° latitude and at 800 to 900 ft. elevation above sea level. Its climatological character is the same as that of the whole Ohio River basin, and experience throughout the basin is therefore pertinent testimony.

Storm Abstracts—The rainfall records of all storms in the weather records of the United States east of 103° longitude were abstracted. All rainfalls were noted down that exceeded the following limits: 1-in. precipitation in 24 hr., or 4 in. for the total storm, where the normal annual rainfall is below 20 in.; and 10% of the annual total in one day, or 15% of the annual total in the whole storm, for stations whose normal annual rainfall exceeds 20 in.

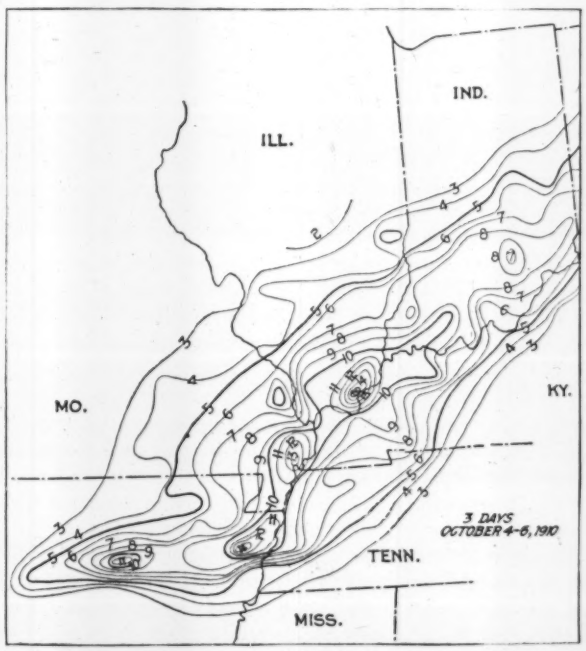


FIG. 3. THE GREAT STORM IN SOUTHERN ILLINOIS, OCT. 4 TO 6, 1910; THREE-DAY RAINFALL DEPTH CURVES

Each station (some 3,000 observing stations were included) had a separate sheet, where besides the daily rainfall were given for each separate storm the one-day total, the two-day and three-day accumulations and sometimes four-, five- and six-day accumulations.

Districting the Country—For easier orientation the whole country (east of 103°) was now districted into 2° quadrangles (divided on the odd degrees); they numbered 133. Designating the rows by numbers and the files by letters, each quadrangle (as 9 E for the Miami) could readily be recognized with respect to its neighbors. Stations on each quadrangle were designated by a, b, c etc., progressing southwest from the northeast corner. Thus, station 9 E z is in the neighborhood of 10 F a. Marking the station sheets accordingly, ready grouping of the storm rainfalls of separate stations into single storms was possible.

A "storm index card" was made for each storm—about 3,500 in all—by bringing together the corresponding precipitations shown by adjoining stations, as traced out through the system of designation just described. This paved the way for drawing storm maps, to furnish a basis for time-area-depth studies.

Time-Area-Depth Studies of Storms

Half of all the storms indexed were found to be one-station storms. Two- to six-station storms amounted to 35%, while 15% covered more than six stations. The latter received nearly all of the subsequent study.

It is necessary to remember that the point of importance now was not the greatest rainfall intensity, nor the greatest 24-hr. precipitation at one station, but the

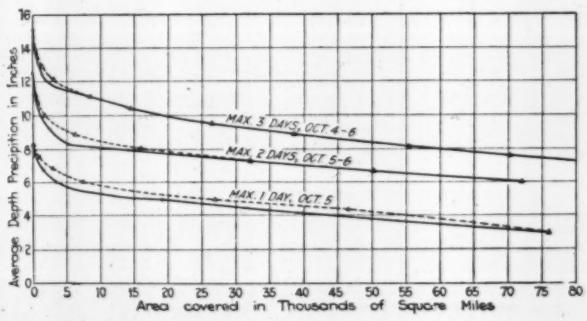

Average Depth Precipitation in Inches
MAX. 3 DAYS, OCT. 4-6
MAX. 2 DAYS, OCT. 5-6
MAX. 1 DAY, OCT. 5
Area covered in Thousands of Square Miles

FIG. 4. TIME-AREA-DEPTH CURVES OF RAINFALL FOR SOUTHERN ILLINOIS STORM

greatest accumulated mass of precipitation over a large area during a period of several days. Storms had to be compared on the basis of the three factors of area of storm, duration, and accumulated depth of rainfall. Drawing contour maps of accumulated rainfall was found

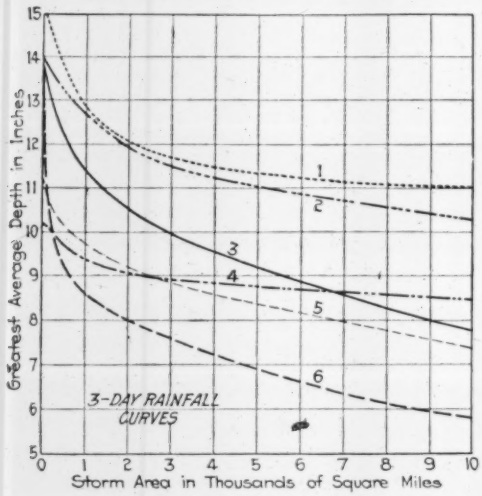


FIG. 5. THREE-DAY AREA-DEPTH CURVES FOR SIX GREATEST STORMS IN THE CENTRAL BASIN

1—Southern Illinois, Oct. 4 to 6, 1910. 2—Arkansas, Aug. 17 to 20, 1915. 3—Iowa, July 14 to 16, 1900. 4—Ohio, Mar. 23 to 27, 1913. 5—Kansas, July 5 to 8, 1909. 6—Michigan, July 19 to 22, 1909.

to be the best means of deriving the curves needed for the comparison.

Only storms covering at least 500 sq.mi. and with maximum intensity of at least one-fifth the total annual rainfall were selected for mapping. Of 46 such storms, the 27 greatest were finally chosen. The nature of the maps is illustrated by Fig. 3, showing the southern Illinois storm of Oct. 4 to 6, 1910; the three-day map is given, those for other lengths of time being similar in general appearance.

The time-area-depth curves for the same storm are shown by Fig. 4, which includes the curves for one to three days. Similarly, Fig. 5 shows in grouped arrangement the three-day area-depth curves for six storms, of which five occurred north of the Ohio River and east of the Dakotas. These (and three others that reached a limit at two days) are the great storms of the Ohio River basin, a region which is a unit as concerns factors of climate and storm incidence. Three of them were much heavier three-day storms than that of 1913[1], and these obviously were possible storms for the Miami Valley.

The storm records showed the general fact that the maximum intensity does not occur at the start of a storm, nor at the end. The heaviest day's rainfall in any storm was found likely to be about half the total of the storm. On the basis of these facts the general shape of the flood-runoff curve (Fig. 2) already began to outline itself.

Long-Time River Records

The storm records thus compiled cover only a moderate period of years, but a sufficient length of time to include several very exceptional occurrences. The question of whether the possible maximum could perhaps be *very much higher*—that is, whether a wholly abnormal great storm might occur some time—was cleared up by a study of old river-flow records.

Ohio River stages at Cincinnati are recorded for 130 years. The Seine at Paris has records for 300 years, the Danube at Vienna for nearly 1,000 years, and the Tiber at Rome for 2,000. In none of these records was any indication of periodicity found. They tell, further, that the greatest flood in a thousand years does not very largely exceed the flood of a magnitude recurring about once in 50 years.

All great floods come within a few feet of each other, and "there has been no great exceptional flood that brought down half again or twice as much water as the other great floods."

The conclusions that could be drawn from the facts developed were expressed very concisely by Chief Engineer Arthur E. Morgan in his testimony before the Conservancy Court a few weeks ago:

> The 1913 flood was one of the great floods of centuries, but in the course of three or four hundred years we may get a flood 15 or 20% greater. We think that is possible, but do not believe we will ever get a flood greater than 20 or 25% in excess of the 1913 flood. But as stated before, we have a factor of ignorance against which we must provide, and the only way we can do this is by arbitrarily increasing the size of the maximum flood provided for. If we had longer records, we could estimate closer; but we believe that in planning works on which the protection of this valley depends, we must go beyond our judgment in the matter. We have done this on every other phase of the design, and we believe that it will not be good engineering practice to stop at our judgment on this phase. We must be able to say that our engineering works are absolutely safe in every respect, and for this reason we have gone 15 to 20% beyond what we believe to be the greatest possible storm, and have provided for one 40% greater than that of March, 1913.

To secure further evidence bearing on the conclusions, the storm records already compiled were analyzed to get the average time-interval of storms of a given size.

Time-Incidence of Storms and the Maximum Long-Time Storm

The underlying idea of the frequency studies was that the storm experience of one station or one quadrangle adds to that of others. The total experience-period, therefore, is the sum of the separate experience-periods. In detail, two methods were pursued—short-time and long-time.

Method A—For each quadrangle the years of record for the several stations were summed; also, the total number of occurrences of a given storm intensity was counted. Dividing the former number by the latter gave the years interval for that particular intensity of storm.

By carrying out this process for a one-day storm intensity equal to 10% of the annual rainfall, and for successive ½-in. additions to this amount, a one-day frequency curve for the quadrangle was constructed. Similarly, a two-day curve was drawn, starting with a fall of 15% of the annual in two days. Three- and four-day curves were drawn where possible.

Fig. 6 reproduces the three-day frequency curve for the Miami quadrangle (9 E). It is evident that the desired maximum storm value is the greatest ordinate of this curve extended far to the right; and the maximum based on all experience in the same climatological region, not merely the Miami experience, would be found by getting the extreme ordinate of a similar curve drawn from the aggregate record of all the quadrangles.

[1]The 1913 storm totaled 7.6 in. on the Miami area, but was heavier a little to the west. The maximum fall on an area as large as the Miami, but lying partly outside, was 9 in. in three days.

Verification of the general shape of the curves is afforded by Fig. 7, a set of frequency curves computed for selected stations along the belt extending from Texas northeast to Maine, frequented by low-barometer areas. These curves, with others computed for the fortieth-parallel belt and for a north-south Mississippi Valley belt, reveal the existence of several distinct types of curve. Among these the Central States type is quite definite and flattens off early. This type is the significant one for floods and flood protection in the Miami Valley.

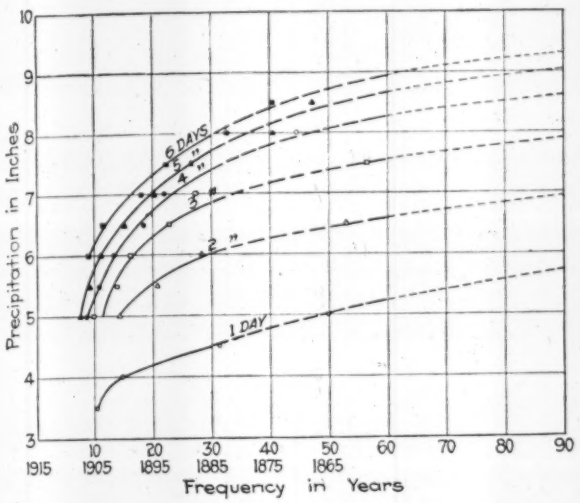


FIG. 6. QUADRANGLE FREQUENCY CURVES; THREE-DAY CURVES FOR MIAMI QUADRANGLE

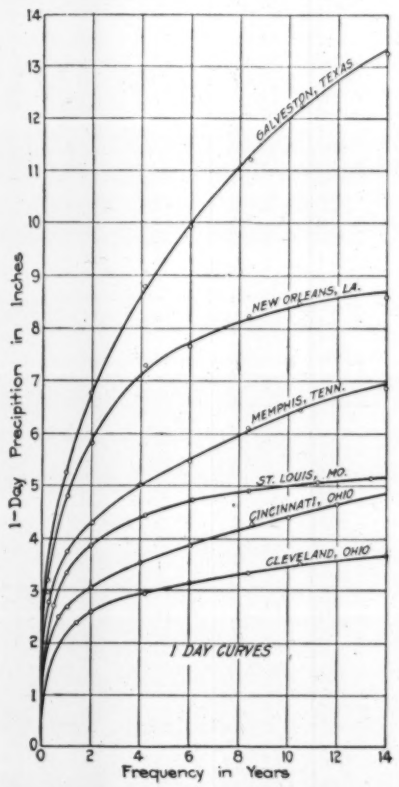


FIG. 7. SHORT-TIME FREQUENCY CURVES FOR ONE-DAY STORMS AT SIX STATIONS IN LOW-TRACK BELT

The method used for these curves gives slightly higher values than that used for Fig. 8, the latter being more correct. The 50-year one-day storm for Cincinnati totals 5½ in.

Long-Time Storms; Method B—It was desired to compute the intensity of the storm that would occur only once in 50 years and once in 100 years, on the average. For this purpose the sum of the record-years of all the stations in a quadrangle, as 360, was divided by 50. The quotient (7.2) represented a number of storms, say seven. Selecting the seven largest storms in the record of that quadrangle and averaging their intensities gave the average intensity of the 50-year storm. Similarly, averaging the four largest storms gave the intensity of the 100-year storm.

This was done for all the 133 quadrangles. The only limitation was that the record at a single station must be at least 10 years.

Fig. 8 gives the 100-year three-day storm-intensity map of the eastern United States, whose data were computed by the process described. These values are close approximations to the ultimate values, for storms of three-day concentration period. Comparison of 100-year and 50-year storms shows that the maximum is closely approached.

The average ratio of increase of the 100-year storm over the 50-year storm, as shown by these figures, is 12 to 15%. This means that the 100-year value is not far below the maximum. Projecting the 50- and 100-year values forward into the future along the curve of the type shown by Fig. 6, the most liberal allowance will give an ultimate maximum storm value about 10 or 15% in excess of the 100-year value.

Another Way of Summing the Experience of the Central Basin

A further development of the experience summation led to results that bring together the entire experience of the Central States. In this way, storm-incidence for very long periods of time could be studied, in spite of the fact that the records were for a short time only.

The area covered by this computation includes practically all of Illinois, Indiana, Kentucky and Ohio, and parts of Missouri, Iowa, Wisconsin, Michigan, West Virginia and Virginia—being the rectangle lying between 37° and 43° latitude and 81° and 91° longitude. There were records from 478 stations in this area, with record lengths ranging from 10 years to 73 years and averaging 15.8 years.

Fig. 9 shows the curves derived from these records. They indicate a slowly decreasing rise to a maximum, but a much slower rise than in Fig. 6 or 7.

The conclusions drawn from the curves were primarily applicable to areas as large as the entire Miami basin. To get values for the separate tributaries (with reference to the individual retarding basins), it was not thought desirable to attempt further mathematical deduction. The developed runoff curve (heavy full line in diagram at left (Fig. 2) was taken as applicable to the two large basins (Huffman and Taylorsville); a slightly higher curve (dash line) was drawn for the three smaller basins.

The Rainfall-Runoff Relation

The studies as described were studies of rainfall. But runoff is less than rainfall, in variable ratio. Does present or future variation of its ratio affect the Miami flood problem? Can higher runoff occur, so that a given rainfall will produce a greater flood?

The 1913 crest flow is estimated at 84% of the maximum rainfall. The drainage area is so widely agricul-

tural that even with bare ground and long rainfall a 100% rate was not reached. Future increase of runoff ratio by improved water-shedding property of the area was considered.

Most of the area is cleared, and its low portions are fully ditched. Tiling to replace ditch drainage is just beginning. Ditches give rapid surface runoff, but leave

FIG. 8. THREE-DAY 100-YEAR STORM MAP OF THE EASTERN UNITED STATES

The contours and figures represent the total rainfall of the three-day storm whose average period of recurrence is 100 years

the ground saturated. Tile drainage carries the water away much more slowly, but continues the process until the excess of water is removed from a 2- or 3-ft. depth of soil, leaving a large temporary storage capacity. These reasons make it probable that extension of tiling will decrease the runoff ratio rather than increase it. The conclusion is confirmed by measurements of runoff from pumping drainage districts in Illinois.

The Seasonal Influence and Snow

While it was concluded that the runoff ratio cannot change for the worse, the chance that a great storm may find frozen ground was taken into account. In connection herewith, seasonal differences were studied attentively. Both rainfall and runoff vary with the seasons, but in inverse sense. The conclusion from a consideration of all the factors was that, with respect to long-time possibilities, the seasonal variations are largely compensatory.

Against the fact that in winter and spring the runoff ratio is greatest—with saturated ground and full discharge from the groundwater reservoir—there is the striking fact that winter storms are not only rare, but also less intense than summer storms. The curves of Fig. 5 show only the 1913 storm as coming in months of high runoff ratio. All the great storms in the area studied include only one other really great winter storm (Dec., 1895, Missouri, 8.78 in. in three days), but this occurred in a region of higher normal rainfall than Ohio.

The judgment of the engineers was that the differences between winter and summer storms and the possibility of frozen ground in late winter make it most reasonable to assume a runoff ratio about the same as that of 1913—

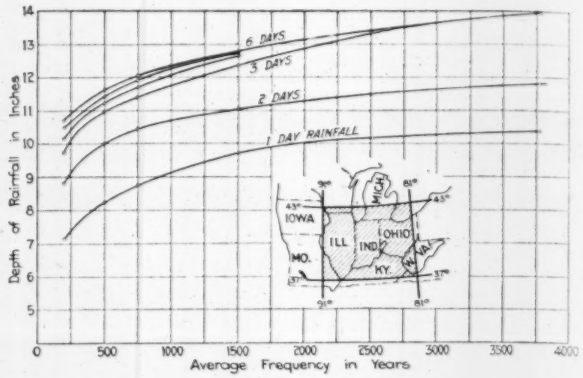


FIG. 9. AGGREGATE FREQUENCY CURVES FOR CENTRAL STATES BASIN

From records of 478 stations having an average period of record of 15.8 years. Curves show frequency of maximum rainfall

namely, 85 to 90%—and therefore to base all assumptions of flood flow directly on comparison with the 1913 flood.

Snow as an additive element was—after full consideration—left out of account. There is no record of a great storm that fell on snow-covered ground. The nature of the air movements that produce long-continued storms is incompatible with the presence of snow or ice, it is held.

Storms in Sequence, and Southern Storms

The soil storage in the Miami area during the summer season was very large. Careful experiments showed that it amounts to 2½-in. water depth. This factor constitutes a protection not only against the intense summer storms, but also against the distant chance that two great storms might follow in close sequence—a chance existing only under summer conditions. The soil drains rapidly enough so that in summer a 10-day interval between storms would suffice to prevent any large influence of the first storm upon the runoff of the second.

The tremendous semitropical rainfalls of the Southern States might suggest startling possibilities for the Miami Valley. The recent (July, 1916) Carolinas storm presents a serious picture, as see the three-day rainfall curves in Fig. 10 (see also the maps of progress of this storm

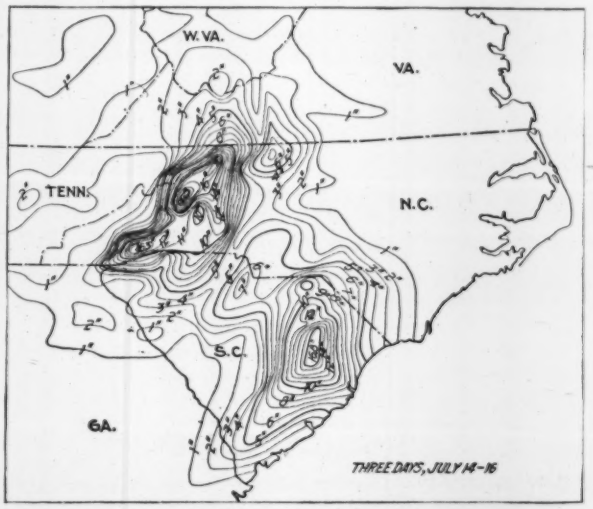


FIG. 10. THREE-DAY RAINFALL MAP OF THE CAROLINAS STORM, JULY 14 TO 16, 1916

printed in *Engineering News*, Nov. 9, 1916, p. 887). But the fact is that such storms cannot cross the mountain barriers; their main burden of cloudwater must discharge as they travel up the slope. The Carolinas storm, for example, showed a tremendous total—18 in. of rain in two days on a 500-mi. area—in North Carolina; yet on the landward side of the mountain ridge, but a few miles west, in eastern Tennessee, only 1 in. of rain fell during the same period.

The storm studies previously described comprise the operation of all causes, including those which can bring Gulf or West India storms up to the Miami Valley. The question of how much water such a storm can deposit near its place of origin, or in the coast states, is not pertinent. The important matter is how much it can bring to Illinois or to Ohio. These studies have already given the answer: Southwestern Ohio can never experience a storm runoff in three days greater than 9½ or 10 in., for an area equal to the Miami Valley.

The studies of rainfall and runoff have been carried out under the personal supervision of Arthur E. Morgan, chief engineer, and S. M. Woodward, consulting engineer; G. H. Matthes has taken charge of working up the rainfall data, and Ivan E. Houk has had charge of the runoff and rainfall measurements in the Miami Valley.

Large Concrete Water Tank Forms Part of Garage

By Herman Myers*

A reinforced-concrete water tank of somewhat unusual construction has recently been built as a part of the garage on the estate of Garret A. Hobart, Jr., Manchester Township, N. J. It forms the central ornamental tower to the garage buildings and the lower part, below the 30,000-gal. tank, is to be used as a chauffeur's room.

The tank is octagonal, having an inside diameter at the bottom of 14 ft. 4 in., this diameter continuing to a height of 12 ft. 8 in. where the vertical walls slant, making the inside diameter at the top 7 ft. 8 in. and having a depth of water at its center of 26 ft. 2 in. The vertical walls vary in thickness from 8 in. at the bottom to 6 in. at the top.

*With John Westerveld, General Contractor, Paterson, N. J.

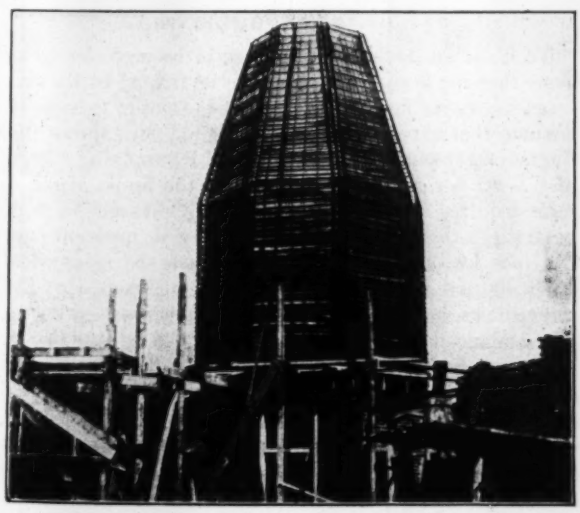


FIG. 1. VIEW OF TANK DURING CONSTRUCTION

The tank is supported by eight reinforced columns extending below grade to solid rock. Their vertical reinforcement comprises six 1-in. square corrugated bars in each column, four of these bars extending vertically to a height of 16 ft. The remaining two bars extend the full height of the tank projecting from the top of the walls, thus providing the necessary anchorage for the tank cover. The vertical reinforcement of the tank walls consists of ⅜-in. square corrugated bars 27 ft. long, placed 12 in. c. to c.

The horizontal reinforcement comprises ⅝-in. square corrugated bars placed 4 in. c. to c. and extending up to a height of 12 ft. 8 in., then ½-in. square corrugated bars placed 4 in. c. to c. extending to a height of 8 ft. 6 in., then ⅜-in. square corrugated bars placed 4 in. c. to c. for the remaining 5 ft. The floor of the tank is 18 in. thick and is of reinforced concrete and hollow tile.

The inside of the tank is covered with ¾-in. cement plaster waterproofed with "Truss-Con" waterproofing paste concentrated, 1 part for every 36 parts of water, and the outside is covered with face brick matching the rest of the building. The sand was obtained from a pit about ½ mi. distant; quarried traprock of size to pass through a ¾-in. ring was obtained from a quarry about 1 mi. distant. The proportions of the concrete for the cover are 1:2:4, and for the floor and vertical walls 1:1½:3.

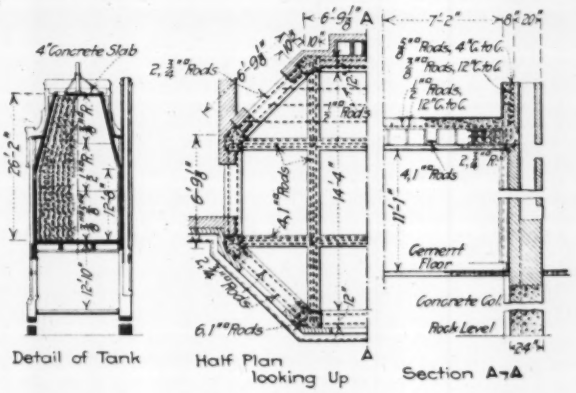


FIG. 2. DETAILS OF CONCRETE WATER TANK

The cover of the tank is a 4-in. reinforced-concrete slab, the reinforcement consisting of ⅜-in. square corrugated bars 8 in. c. to c. both ways.

The design was made by and the work was done by John Westerveld, Contractor, Paterson, N. J.

Asphalt-Paving Statistics—Dec. 11, 1916, is the fortieth anniversary of the asphalt-paving industry in the United States. On that date in 1876 there was completed the first extensive asphalt pavement, on Pennsylvania Ave., Washington, D. C., from Sixth St. to the Treasury Building. This work was carried out with asphalt imported from the Island of Trinidad. The avenue was paved in accordance with ideas developed by E. J. de Schmedt, a Belgian chemist. Since 1876 the yardage of natural-asphalt pavements has increased from the 54,000 sq.yd. laid on Pennsylvania Ave. to the amount of about 200,000 sq.yd., or roughly the equivalent of 22,000 mi. of city streets, the larger part of which have been constructed with asphalt from the Island of Trinidad, according to the Barber Asphalt Co., which compiled the statistics.

Activated-Sludge Power Costs

By Gustav J. Requardt*

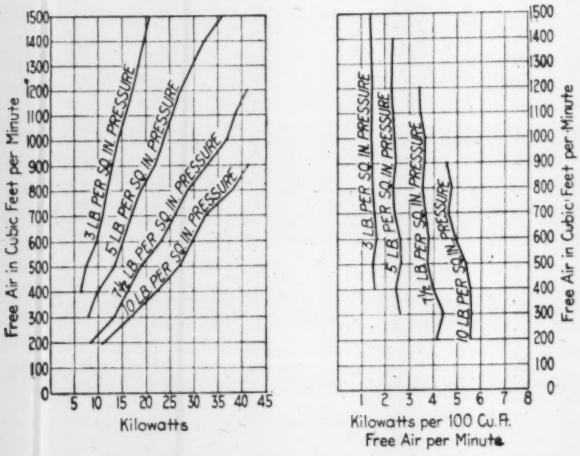
Given the amount of air delivered by a blower (requiring a definite amount of power for operation), the ratio of air to sewage necessary for the degree of treatment desired and the unit cost of power, it is a simple calculation to find the power cost for sewage treatment by the activated-sludge method. When many calculations are to be made at one time, the writer has found that much work may be eliminated by the construction and use of a set of curves in which all factors are considered to vary within limits suited to average conditions.

Ezra B. Whitman recently instructed the writer to draw up such a set of curves for use in his office, taking the relations between necessary power and delivered air from the catalogs of various air blower and compressor manufacturers. These curves are herewith presented. While data from only one manufacturer of air blowers (the Connersville Blower Co.) are given, it is a simple matter to plot on the same sheet the values as given for the several other makes of air blowers and compressors.

Construction of Diagrams

Fig. 1 is made by plotting as abscissas the power necessary to drive the machine, and as ordinates the corresponding free air delivered, there being separate curves for various air pressures. Plotting thus the information taken from the catalogs of the various makers, a quick comparison of efficiencies may be made.

Fig. 2 is made up from Fig. 1. The ordinates remain the same (free air delivered) while the abscissas are obtained by dividing the power by the corresponding volume of free air in units of 100 cu.ft. For example: At 5 lb. per sq.in. pressure a 22-kw. machine will deliver 900 cu.ft. of free air per minute; therefore, 2.4 kw. of power is required per 100 cu.ft. of free air per minute in a machine capable of delivering air at the rate of 900 cu.ft. per min. It is to be noticed that in the type of blower taken as an illustration for these curves the efficiencies but slightly increase as the machines become larger.


Free Air in Cubic Feet per Minute
3 LB PER SQ IN PRESSURE
5 LB PER SQ IN PRESSURE
7½ LB PER SQ IN PRESSURE
10 LB PER SQ IN PRESSURE
Kilowatts
Kilowatts per 100 Cu.Ft. Free Air per Minute

FIGS. 1 AND 2. POWER REQUIRED TO DELIVER FREE AIR

Fig. 1 (Left)—Power necessary to deliver compressed air with Connersville blowers. Fig. 2 (Right)—Power required to deliver 100 cu.ft. of free air per minute

*Assistant to Ezra B. Whitman, of Norton Bird & Whitman, Engineers, Chicago and Baltimore.

Fig. 3 is made by plotting as ordinates the kilowatts per 100 cu.ft. of free air per minute, and as abscissas the kilowatt-hours per million gallons of sewage, the radiating solid lines giving the various proportions of volumes of free air in cubic feet to volumes of sewage in gallons. The same radiating lines also give relations between sewage flow and volume of free air required for treatment. The relations between kilowatt-hours per million gallons of sewage treated and cost of power per million gallons are also plotted on this sheet, the radiating dotted lines representing the various unit costs of power.

The method of proportioning air supply to sewage flow, volume for volume, is far simpler, in the writer's mind,

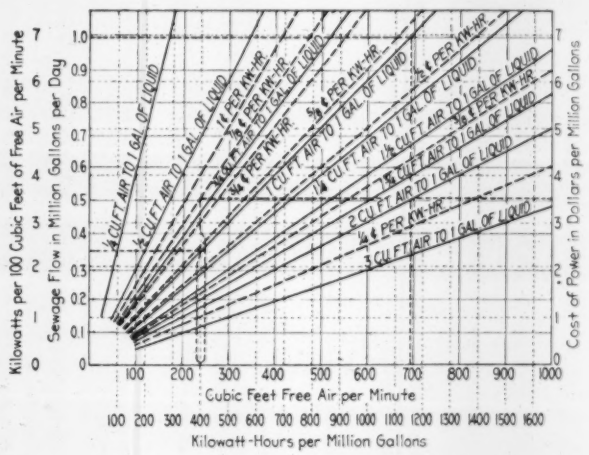

Kilowatts per 100 Cubic Feet of Free Air per Minute
Sewage Flow in Million Gallons per Day
Cost of Power in Dollars per Million Gallons
¼ CU FT AIR TO 1 GAL OF LIQUID
½ CU FT AIR TO 1 GAL OF LIQUID
1 CU FT AIR TO 1 GAL OF LIQUID
1½ CU FT AIR TO 1 GAL OF LIQUID
2 CU FT AIR TO 1 GAL OF LIQUID
3 CU FT AIR TO 1 GAL OF LIQUID
¼ PER KW-HR
½ PER KW-HR
¾ PER KW-HR
1 PER KW-HR
1¼ PER KW-HR
1½ PER KW-HR
2 PER KW-HR
Cubic Feet Free Air per Minute
Kilowatt-Hours per Million Gallons

FIG. 3. RELATION BETWEEN SEWAGE FLOW AIR AND POWER REQUIRED IN ACTIVATED-SLUDGE PLANTS

1,000,000 gal. per day = 694 gal. per min.

than any other method. Some engineers prefer to state the volume of air supply to the area of tank surface per unit of time. This brings the dimensions of the sewage tank or container and the length of time of air treatment into the problem, where these elements do not belong; they can be much better handled in the actual design of the plant. It is well to remember, then, in using the curves, that *volume* of air supply is proportional to *volume* of sewage treated, irrespective of the size or shape of tank or of the length of time of air agitation and treatment.

How To Use the Curves

To illustrate how the curves are to be used, let us assume that the sewage of a city is to be treated by the activated-sludge method and that the cost of power is required. Assume that experiments on the sewage have shown that the necessary purification is obtained by applying 1 cu.ft. of free air per gallon of sewage; that the proper depth of tank requires air to be delivered at a pressure of 5 lb. per sq.in.; that current in the locality in question costs ⅞c. per kw.-hr. The sewage flow may be assumed at 1,000,000 gal. per day. It is to be noted here that, for larger flows, a straight-line relation holds between sewage flow and air required and that the air can be supplied by any number of blowers or compressors. Larger problems can thus be split down to smaller units and each solved separately.

With the elements above assumed having been determined, we are now ready to make use of the curves. Enter Fig. 3 at the ordinate for 1,000,000 gal. sewage flow per day and note where this ordinate strikes the radiating

solid line for 1 cu.ft. air to 1 gal. liquid, the abscissa at this point reading 694 cu.ft. free air per minute. This determines the size or capacity of air blower required. Now enter Fig. 2 at the ordinate for, say, 700 cu.ft. of free air per minute and note where it strikes the line for 5 lb. per sq.in., the abscissa at this point reading 2.4 kw. of power per 100 cu.ft. of free air per minute. Again using Fig. 3, note where the ordinate for 2.4 kw. per 100 cu.ft. air per minute strikes the radiating solid line for 1 cu.ft. air to 1 gal. liquid. The abscissa at this point reads 400 kw.-hr. per million gallons of sewage treated. Reading upward along this abscissa, note where it strikes the radiating dotted line for 7/8c. per kw.-hr. The ordinate at this point, toward the right-hand margin, reads $3.50 cost of power per million gallons of sewage treated, which is the result desired.

In the example given above, no recognition has been taken of the fluctuation of sewage flow as it reaches the plant. To apply to each gallon of sewage its proper volume of air, a detention chamber can be utilized so that the raw sewage may be passed into the agitation compartment of the activated-sludge tank at a uniform rate; or the capacity of the air-blower units can be figured upon the maximum or peak flows with the idea that separate units are to be shut down, one by one, as the sewage flow decreases.

Flow of Air Through Orifices Against Back Pressure

By B. S. Nelson*

The flow of air through orifices exhausting at atmospheric pressure has been studied and formulas have been deduced applicable to that condition, but there seem to be few data available as to the amount of air that will flow through a given size of orifice against a back pressure, with a definite initial pressure and final pressure.

Herein are given the results of an attempt to find an empirical formula to fit such cases. The formula found,

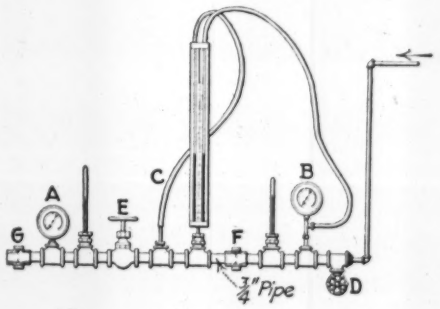


FIG. 1. APPARATUS USED FOR AIR ORIFICE EXPERIMENTS

while tried for a special case only—and perhaps it is a special case of some known formula and unrecognized as such—is interesting nevertheless for its simplicity and for the fact that it checks well for the whole range of the experiments.

The flow of air through an orifice against a back pressure is of interest in connection with the design of air nozzles for air-lift pumping from deep wells, and the experiments made had in view a formula applicable to cases where the orifice exhausts against a back pressure slightly lower than the initial or receiver pressure. It would also be useful for measuring air through an orifice in a pipe line where only a slight drop is desirable, as, for example, a meter.

The apparatus used, Fig. 1, was made up principally of standard 3/4-in. pipe valves and fittings. The orifice plates were held between the two halves of the unions shown, which were faced in a lathe; a gasket was put on each side. The orifice plates were thin steel disks, 1/16 in. thick, with several sizes of holes drilled and the burrs honed off on an oilstone.

The plates marked *F* (referred to as the measured orifices) were the subjects of the experiments the plates *G* being used simply as meter orifices to measure the air that had passed through *F*.

Three sizes of measured orifices were used—1/32, 3/64 and 1/16 in.—and the same three sizes of meter orifices were used. The object of having different sizes of meter orifices was, first, to best suit the discharge of the particular size of hole

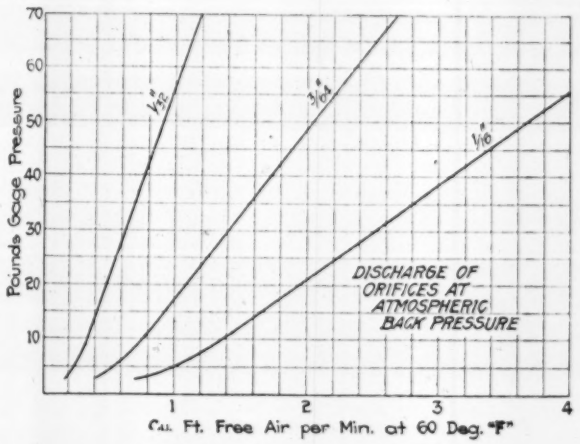


FIG. 2. DISCHARGE THROUGH ORIFICES AT ATMOSPHERIC BACK-PRESSURE

tested and, second, to check by taking readings on the same measured orifice with different meter orifices.

The object of the two thermometers shown was to make temperature corrections for the cooling due to expansion through the orifice *F*; but it was found that this drop was negligible, so the thermometers were not used.

Gages *A* and *B* were calibrated with a dead-weight tester, and corrections were applied to results. In order to measure the drop in pressure through *F* accurately, a mercury U-tube or manometer was used, graduated to tenths of inches and connected by tubing to each side of the orifice.

The apparatus was connected to a tank containing air at fixed pressure; by regulating the bypass valve *D* and the valve *E* any initial pressure and any drop desired through the orifice *F* could be obtained.

The tail pressure on the meter orifice *G* (which was of course different from either of the above) was an index to the amount of free air escaping to the atmosphere. This amount was the same that had passed through the measured orifice and was the result sought.

The air discharge was computed from the formula

$$W = \frac{0.53AP}{T}$$

where

W = Weight of air in pounds per second;

*7319 Panola St., New Orleans, La.

P = Pressure behind orifice in pounds per square inch absolute;
A = Area of orifice, square inches;
T = Absolute temperature.

As a matter of convenience the discharge at various pressures for each of the three sizes of meter orifices was plotted graphically, as shown in curves, Fig. 2.

By manipulating valves D and E a group of observations was made with a certain fixed pressure behind orifice F and several different pressure drops through it, the range of drop being as great as possible with the apparatus. This was repeated with several (usually four) different pressures behind F.

TABLE 1. SET OF READINGS TAKEN ON ¼-IN. ORIFICE, WITH ¼-IN. METER ORIFICE

Gage B, Lb.	Gage A, Lb.	Discharge, Cu.Ft. per Min.	Drop through F, Lb.
90	14 45	0 92	2 66
90	10 80	0 80	1 97
90	7 80	0 68	1 48
75	32 50	1 48	7 52
75	26 40	1 28	5 66
75	18 0	1 02	3 59
75	11 0	0 81	2 17
60	31 2	1 44	8 40
60	25 9	1 27	6 50
60	19 4	1 07	4 57
60	12 0	0 84	2 65
45	26 0	1 27	8 46
45	21 5	1 13	6 50
45	15 9	0 96	4 42
45	9 8	0 76	2 66

The size of meter orifice was chosen by preliminary trial, to best suit the range of the particular set of observations; at the end of the experiment one or two readings were repeated with a different meter orifice as a check. It was found that these checked readings agreed within about 5%, which was deemed close enough for practical purposes.

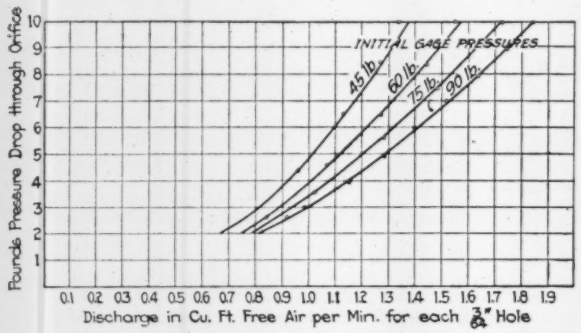

Pounds Pressure Drop through Orifice
INITIAL GAGE PRESSURES
45 lb.
60 lb.
75 lb.
90 lb.
Discharge in Cu. Ft. Free Air per Min. for each 3/64" Hole

FIG. 3. POINTS IN THESE CURVES WERE PICKED FOR PLOTTING CURVES IN FIG 4

In order to reduce all the readings to convenient form, it was necessary to plot the actual readings graphically and then from these curves pick points from which to plot a second set of curves with the desired ordinates and abscissas. The curves in Figs. 3 and 4 illustrate this for one size of orifice, and similar curves were made for each size of orifice.

Each of the curves in Fig. 3 is for a separate initial pressure P and is plotted with drop in pounds through the orifice F as ordinates, and cubic feet of free air discharged per minute per hole as abscissas.

From these curves points were picked to plot the final curves in Fig. 4, which are in more convenient form. For instance, if one wanted to know the number of $^3/_{64}$-in. holes to put in an air-lift nozzle, with 80-lb. receiver pressure and 5-lb. drop through the nozzle: Using Fig. 4, start at 80 lb. on the ordinate line, run over to the curve for 5-lb. drop and down to the abscissa line; we get 1.24 cu.ft. of free air per minute discharged per hole, which divided into the total quantity of air gives the total number of holes required.

It was suspected from the smoothness of these curves that there probably was a formula that would fit every point in the range of the experiments. The discharge should vary directly as the area of the orifice and as some power of the two variables—initial pressure and drop through the orifice—and there would also be some constant K.

It was assumed from other air formulas that this power of the initial pressure was the ½ or square root, and using this in a number of random readings it was found that in each case the same power of the drop in pressure (the ½ power) held good. The result was the formula

$$Q = D^{\frac{1}{2}} \times P^{\frac{1}{2}} \times A \times K$$

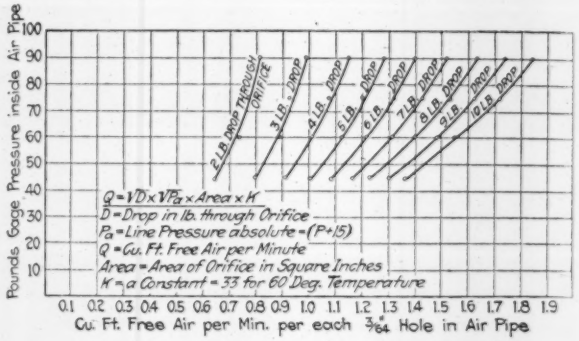

Pounds Gage Pressure inside Air Pipe
2 LB. DROP THROUGH ORIFICE
3 LB. DROP
4 LB. DROP
5 LB. DROP
6 LB. DROP
7 LB. DROP
8 LB. DROP
9 LB. DROP
10 LB. DROP
Q = √D x √Pa x Area x K
D = Drop in lb. through Orifice
Pa = Line Pressure absolute = (P+15)
Q = Cu. Ft. Free Air per Minute
Area = Area of Orifice in Square Inches
K = a Constant = 33 for 60 Deg. Temperature
Cu. Ft. Free Air per Min. per each 3/64 Hole in Air Pipe

FIG. 4. FINAL OR WORKING CURVES

where

Q = Discharge in cubic feet free air per minute;
D = Drop through orifice in pounds per square inch;
P = Initial pressure, pounds absolute;
K = Constant whose value is 33 for 60° F.

The value of K was obtained by computing it from points taken from the curves in Fig. 4. These points, to obtain a good average, were selected for drops of 2, 4, 6, 8 and 10 lb., with initial pressures of 45, 60, 75 and 90 lb., making a total of 20 values for K for each orifice or 60 for the three. A set of these values is given in Table 2. The average for the $^1/_{32}$-in. orifice was 33.1, for the $^3/_{64}$-in. was 32.3, and for the $^1/_{16}$-in. was 33.6, or an average for the three of 33.

TABLE 2. VALUES OF K CALCULATED FROM POINTS TAKEN FROM CURVES IN FIG. 4

Orifice	Line Pressure, Lb. Absolute	Drop in Pounds through Orifice 2	4	6	8	10	
$^1/_{32}$-in.	60	32.1	31.4	32.4	33.2	33.6	
	75	31.9	29.5	32.7	33.0	33.5	
	90	31.6	30.0	33.0	33.2	33.2	
	100	33.1	31.8	31.1	33.1	32.8	Average, 32.3
$^3/_{64}$-in.	60	34.3	34.1	33.2	32.4	32.4	
	75	34.3	33.8	33.0	33.0	33.9	
	90	33.6	33.0	32.8	33.6	33.1	
	100	32.2	32.5	32.2	32.6	32.8	Average, 33.1
$^1/_{16}$-in.	60	34.9	33.7	32.6	32.2	32.0	
	75	34.7	33.4	33.0	32.9	33.2	
	90	34.2	33.5	33.4	33.6	33.9	
	100	33.6	33.4	33.6	36.2	34.2	Average, 33.6
General average							33.0

The formula has not been checked for ranges beyond those on the curves, nor has the value of K been found for temperatures other than 60°, because these served the purpose for which the experiments were made. It would be interesting for someone with proper laboratory facilities to extend the experiments for greater range of drop and to find the temperature coefficient.

Rebuilding with Permanent Surface Old Macadam Roadways

First-Prize Road-Building Article in the "Engineering News" Prize Contest

By Charles A. Carruth*

The division office of the New York State Department of Highways, Rochester, N. Y., started in 1909-10 some very interesting and profitable experiments, in the resurfacing of worn-out macadam roads, with permanent or semipermanent pavements.

These experiments, the first of their kind under the Commission of Highways, were begun under former Division Engineer M. W. Wilbur and consisted of resurfacing macadam roads, built years previous, which on account of changed traffic conditions were becoming expensive to maintain as waterbound macadam. The resurfacing materials used were brick, bituminous concrete (Topeka and "Amiesite"), rock asphalt, McClintock's concrete and brick cubes, "Rocmac," and bituminous macadam, penetration method.

As some of the above-mentioned materials have stood the wear of greatly increased traffic for six or seven years with practically no maintenance, they may be considered under the head of permanent paving, if there is any such thing.

The writer, while not connected with this work in any official capacity but as employee of the Department of Highways at Rochester, was quite familiar with the construction employed, and being a fellow employee of G. G. Miller, engineer in charge of this work, has gathered from him the data necessary to complete this article.

This resurfacing was done on highways leading out of Rochester, two of the roads beginning at the city line and carrying heavy mixed traffic.

Brick on Renovated and Grouted Macadam

The description of the resurfacing of the Scottsville Road, No. 63, follows: The road was built in 1902-03 of waterbound macadam 6 in. thick. The soil is heavy clay with quicksand pockets. This road carries a heavy farm traffic. There is a brickyard located near it, the output of which is carted to the city by teams and motor trucks. Much extensive and expensive repair was continuously necessary, especially near the city line. In 1910 a contract was let to resurface ¾ mi. of this road with a 4-in. brick pavement, using the macadam as a base.

An edging 8 in. thick and 10 in. deep, of second-class concrete, was first constructed to the required line and grade. The macadam was scarified. This material was forked over to allow the dirt and fine material to go to the bottom. New stone of ¾ to 1¼ in. in size was added to make up the necessary grade and crown. The stone was grouted with a mixture of 1 part cement to 2½ parts sand, rolling was carried on while the grout was being applied, until grout flushed to the top of the stone. This base was allowed to set for a few days. A sand cushion 1½ in. thick was then made, the brick placed and grouted with a 1:1 mixture. The cost of brick, cushion, edging and preparation of the base was $1.60 per sq.yd. of brick pavement; to this cost may be added 24c. per sq.yd. to cover widening of embankment, graveling shoulders, drives, etc., not always needed for such resurfacing work. The brick and edging gave a paved width of 17 ft. This road went from 1910 until last year without any repairs, except to shoulders.

As the grouting of the brick was done late in the fall, it must have been injured by frost, for in the spring of 1911 much of the grout came out, causing cobbling of the brick at the joints. In 1915 the pavement was swept clean and an application of Tarvia B, 0.2 gal. per sq.yd., was applied and covered with a thin coat of clean sharp sand. While this tar treatment has worn off in some places, the joints are filled flush and the cobbling has been stopped. There are no cracks or breaks in the pavement, and it should be good for many years with very little maintenance.

Macadam Resurfaced with Bituminous Concrete

The resurfacing of Monroe Ave., Road No. 94, was done in 1910. This road was built in 1902-03 of waterbound macadam and carries heavy mixed traffic. There is a brickyard ½ mi. from the city line. One mile was resurfaced, using two kinds of asphalt and types of bituminous concrete. The old macadam was scarified lightly and brought to proper crown and grade with new stone ¾ to 1¼ in. in size, then well rolled and filled. One-half of a mile was built of Topeka type, using California asphalt. The pavement on this section was 2 in. thick when finished. The mixture was made in a semiportable Koehring paving plant.

The proportion per batch was 2,070 lb. sand, 1,035 lb. stone chips up to ½ in. in size, 945 lb. stone dust and 450 lb. asphalt. Some cracks and pot holes developed in this section; these have been repaired by patrolmen at a cost of about $10. This section is wearing thin, but is good for several years with careful maintenance.

After the old macadam was scarified and brought to grade as described above, the second section, nearest the city, was resurfaced by first laying a binder coat of stone and asphalt 1 in. thick. On this was placed a mixed top, 2 in. thick when rolled, using the following proportions for each batch: 500 lb. sand, 300 lb. stone chips up to ½ in. in size and 88 lb. of Trinidad asphalt, the thickness of surface being 3 in. finished. This section is in excellent shape and has had no maintenance in the past six years. Each type cost $1.24 per sq.yd., including the work on the base, the cost being high on account of the small amount contracted for.

The resurfacing of East Ave., Road No. 5, in 1909 with "Amiesite" at a cost of $1 per sq.yd. was another successful experiment, and after seven years with practically no maintenance it is in fair condition today.

These experiments have stood the ravages of modern traffic conditions and are good for some time to come, showing that with the proper selection of resurfacing materials much can be done to utilize the many miles of macadam that has done its duty as a road surface.

*County Assistant Engineer, New York State Highway Department, Rochester, N. Y.

Concrete Paving for Small Levee on the Wabash River

By George C. Graeter*

The Wabash River levee in Sullivan County, Indiana, was constructed in 1895 at a cost of $74,500. Starting at the Illinois Central R.R. embankment at Riverton, Ind., it extends downstream along the left bank of the river for 8 mi. (in which high land serves as a levee for about 1 mi.) and then about 1.6 mi. up the right bank of the Prather drainage ditch, which is the new channel of Busseron Creek. This levee was constructed to protect some 12,000 acres of farm land from the floods of the Wabash River and Busseron Creek, and in connection with three drainage systems (two county and one private) it virtually reclaimed 7,500 acres of bottom and marsh land that now has an average market value of $100 per acre.

The earth used in the original construction of the major portion of the levee was a friable sandy loam, and the

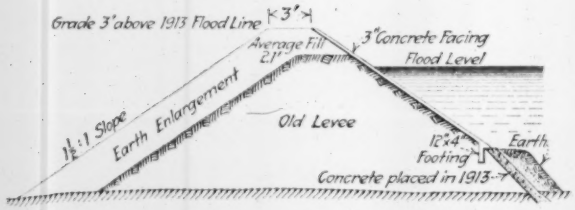


FIG. 1. WABASH RIVER LEVEE, SHOWING EARTH ENLARGEMENT AND CONCRETE FACING

levee as a whole (and more particularly the upper end) has never afforded adequate protection from high water, erosion, seepage or burrowing animals.

In 1904, $1,000 was spent for repairs; in 1912, $18,300 for repairing breaks; and in 1913, $14,500 for repairing breaks and for placing concrete at the river-side toe of slope on 12,500 ft. of the upper end. Maintenance has cost $14,000 (or about $650 per year), most of this being spent in digging out ground-hog and mole burrows and mowing the levee. Thus the cost of repair and maintenance up to 1916 was over 66% of the first cost of construction, and the levee was still in poor condition.

In March, 1916, the Levee Committee (which is elected by the owners of the majority of the land in the levee district) retained the writer to make an investigation of the existing levee and prepare plans for its repair and improvement. The cross-section, Fig. 1, shows the plan adopted. It consists in enlarging the cross-section with material added to the top and the land side and in protecting the river slope by a 3-in. concrete facing or revetment.

The concrete extends for about 12,500 ft. from Riverton to a sand ridge. It consists of 199,100 sq.ft. of facing and 154.3 cu.yd. of footing. The earth enlargement on this upper section averages 231 cu.yd. per 100-ft. station; on the next 20,000 ft. beyond this ridge it averages 163 cu.yd. per station, and then for about 6,700 ft. the average is 240 cu.yd. per station. The total enlargement, including a short stretch of relocated levee, amounts to 88,000 cu.yd.

*Civil Engineer, Sullivan, Ind.

On the section that is being concreted the earth enlargement (28,890 cu.yd.) is being put up by a reconstructed Monighan dragline excavator, shown in Fig. 2. The remainder of the enlargement is being constructed with teams, using slip and wheeled scrapers. The shrinkage allowance was 20% for dragline work and 10% for team work. The top and land slope of the levee were plowed, and all stumps and brush were removed before placing the fill for the enlargement. The Levee Committee provided borrow pits without expense to the contractor. The specifications permitted material to be taken on either side of the levee, but required it to be taken on the river side where possible. No material was to be taken within 22 ft. of the toe of the old slope, and new pits were not allowed to be more than 3½ ft. deep.

Concrete Facing for Levee

The concrete facing for the river slope is a 3-in. slab extending from the top of the levee to the old concrete toe and having behind this toe a 4-in. footing wall 12 in. deep, as shown. This footing is formed as part of the slab. Joints are provided at intervals of 20 ft. by ½-in. wood strips 3 in. deep. Each 20-ft. section is cast as a single slab. When the concrete of two adjacent sections has set, the wood partition strip is removed and the ½-in. space filled with concrete without any attempt to bond the old and new concrete.

The concrete was composed of gravel taken from the bed of the river and having not less than 30% or more than 40% of sand retained on a 64-mesh screen. The coarse aggregate had a maximum size of 1½ in. and was specified to contain not over 5% (by weight) of loam

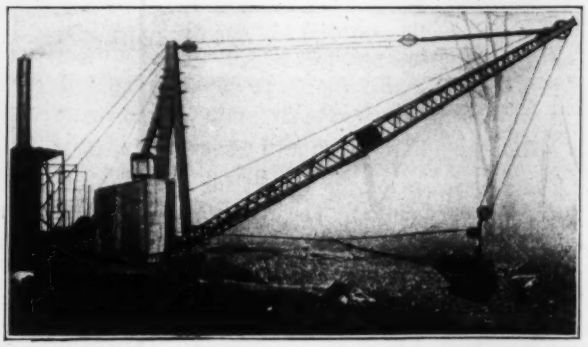


FIG. 2. DRAGLINE EXCAVATOR FOR LEVEE ENLARGEMENT

or other foreign matter. The concrete was moderately wet, proportioned 1:6, and was made in a batch mixer of 7-cu.ft. capacity. An addition of 10% (by weight) of hydrated lime was provided for by the engineer, but not admitted by the Levee Committee.

Deposits of suitable sand and gravel were found on the river side of the levee at the lower end of the concrete work and on a bar in the river at about the middle of the work. For the upper end, the coarse aggregate was hauled from a pit about ½ mi. inland from the levee.

The river face of the levee was grubbed and cleared of all vegetation, and any marked depressions were filled with earth. A plow furrow was run at the back of the old concrete toe, and the trench for the footing was dug with tiling spades. The slope was then dressed, wetted and tamped with a 75-lb. rammer. The mixer was placed

at the toe of slope, and the concrete was shoveled by hand from the discharge platform, as shown in the upper view, Fig. 3. The wheeled cart, holding 6 cu.ft. of aggregate, was pulled up the runway by a wire cable on a drum operated by the engine of the mixer.

The footing was placed first, and then the facing was placed from the toe of slope and footing upward. The surface of the concrete was smoothed with the backs of the shovels, no striker or templet being used. The method of work could be improved by using a mechanical elevator to place the concrete on the slope. Within an hour after the placing of the concrete it was covered with a 1-in. layer of earth, which was thoroughly sprinkled the next day. This covering, which assisted the curing of the concrete, was left in place for about ten days in hot weather. The lower view in Fig. 3 shows the concrete slope partly covered with earth. This view is at Riverton, and the rodman is holding the level rod on a bench mark set at the flood level of 1913 (El. 442.68).

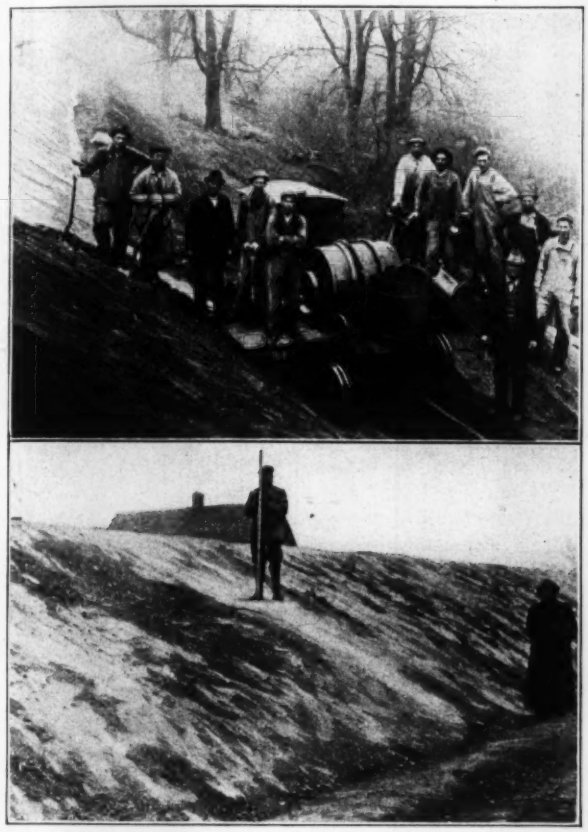


FIG. 3. CONCRETE FACING FOR THE WABASH LEVEE

The upper view shows the mixer plant and gang, and a stretch of finished concrete slope. The lower view shows the finished concrete partly covered with earth. The level rod is being held at the level of the flood line of 1913

Numerous superficial cracks have appeared in the finished facing. This was expected, most of them being in the upper part of the slab, where the concrete is laid on the new earth enlargement. They are probably due to settlement and rapid setting and are not increasing in size or number.

It is expected, however, that the facing will prevent erosion, reduce seepage and prevent burrowing animals from working through the levee, and that in combination with the earth enlargement it will enable the levee to withstand extreme high water without damage to itself or to the land behind it.

Contractors and Cost

The contract was awarded to M. R. Polk & Co., of Oaktown, Ind., at 20c. per cu.yd. for earth enlargement and $6.60 per cu.yd., or practically 6.1c. per sq.ft., for the concrete. The concrete work was sublet to Charles Ridgway, of Carlisle, Ind, and was done by him at a cost of $5.12 per cu.yd., or about 4.75c. per sq.ft. The pay was $2 per 9-hr. day for labor and $4 for teams. The cost of this concrete work was distributed as follows:

	Cost per Cu.Yd.
Cement (cost, hauling and storing)	$2 52
Gravel (purchase price, stripping pits or deposits and hauling)	0 55
Labor and superintendence (preparation of slope, wetting and surfacing, also mixing, placing and finishing concrete)	1 87
Equipment and repairs (counting 50% depreciation and including gasoline for mixers)	0 18
Total cost per cu.yd.	$5 12

The record run for the mixer and gang shown in Fig. 3 was about 3,500 sq.ft. (or about 32 cu.yd.) in one day. An inspector in the employ of the Levee Committee and under the direction of the engineer was on the work all the time. Semimonthly payments were made to the general contractor, on the engineer's estimates, 20% being retained until the completion of the contract.

Shaft Sinking by Freezing Revived in England

Shaft sinking by the freezing process was carried on at least as far back as 1862, but of late years the method has fallen into disuse. The *Engineer* describes a recent application of the freezing process at the new Llay Hall colliery, Wrexham, England. Both the upcast and the downcast shafts—270 ft. apart—which were originally giving off large feeders of water, have been carefully treated and the brickwork made quite dry. The only water that now has to be disposed of comes from the underground workings. The difference in level between the stratification of the two shafts is about 40 ft., corresponding to a dip of 17° from No. 2 to No. 1.

At first, two foreshafts were sunk 12 ft. deep and 31 ft. in diameter and lined with brickwork. Concrete was placed on the bottom after inserting straight vertical guide tubes for 25 bore holes distributed around a circle 26½ ft. in diameter. In the upper strata the holes for the freezing tubes were bored by the rope method, but in the lower and harder ground percussion apparatus operated by a steam cylinder was used.

The cold necessary for freezing the water-bearing ground was applied by two distinct groups of machines. The pipes and valves were arranged so that either group of machines could operate on either shaft or on both simultaneously. Each included a belt-driven ammonia compressor with an independent horizontal steam engine having cylinders 16¾ in. in diameter and 30-in. stroke. A solution of calcium chloride was used as the freezing liquid and was caused to circulate between the refrigerators and the freezing tubes by means of two plunger pumps. The brine pipe between plant and shaft was insulated with hair felt.

The ground was frozen to a depth of 231 ft. in the first and 235 ft. in the second shaft. The diameter of the pit shafts after lining is 18 ft.

Placing Ornamental Concrete on Steel Frame

By Harry L. Dlyn*

SYNOPSIS—Details of costs and methods in placing concrete casing on rather complicated steelwork of a new elevated station just completed for the rapid-transit enlargement in New York City.

The Pelham Parkway station of the White Plains Ave. elevated extension to the old subway lines of New York City is located at the intersection of White Plains Road and Pelham Parkway and about 1,000 ft. north of the Bronx and Pelham Parkway entrance to Bronx Park. It was decided to enchance the appearance of this structure and make it an ornamental concrete station in harmony with the pleasing surroundings of this section, with landscape forming a beautiful foreground for the architectural effects of an ornamental structure.

The station is of the mezzanine type, the tracks being carried over a mezzanine story at each end of the structure in through girders to allow sufficient headroom for the mezzanine floor. The structure is 550 ft. long over nine spans, symmetrical about both center lines. The steel columns for the track girders are situated in the street, and the station platforms are carried on cross-frames framed into steel lattice sidewalk columns and lattice wall girders, both incased in concrete.

The principal features of the exterior architectural effects are the massive appearance of the structure, the

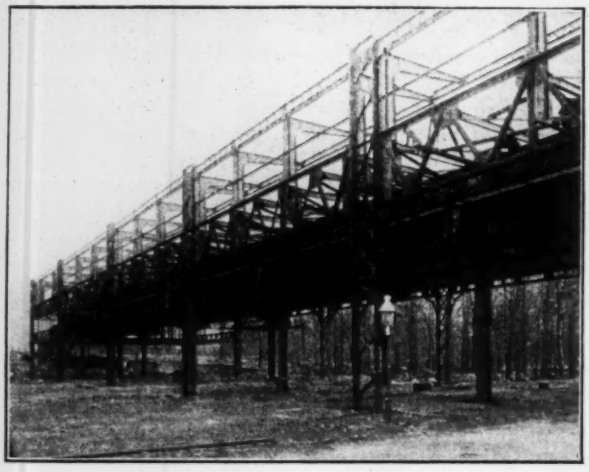


FIG. 1. STEEL FRAME OF PELHAM PARKWAY STATION, WHITE PLAINS ELEVATED R.R.

unbroken and graceful lines of the walls and brackets merging into the columns, the exterior being set off with inlaid colored tile to add brightness to the natural concrete finish of the structure. The entire appearance of the station harmonizes with the purpose of the structure, and its severe structural character shows an expression of strength, magnitude and dignity, the keynote being simplicity in every respect, with the costly architectural treatment reduced to a necessary minimum.

*Engineering department, Public Service Commission, New York City.

Well-seasoned timber of an accepted grade was used for the formwork. The forms were cut and laid out on the ground in sections and then hoisted up to be set in place. The contractor used an all-electrical equipment, which included an electrically operated saw.

Great care was employed in erecting the forms and keeping them in proper alignment during the entire process of construction. Owing to the fact that the concrete is open to view, experienced carpenters were

FIG. 2. CONCRETE CASING JUST AFTER FORMS WERE REMOVED

employed, as only first-class help could handle efficiently this quite complicated piece of work.

It developed that the sludge oil which was used for the forms stained the concrete, and therefore oiling the forms was later eliminated. Instead, the forms were made wet, previous to pouring the concrete, by playing a hose upon them freely. This proved an effective means of making the forms water-tight, preventing the cement from leaking through the cracks in the wood and making a smooth uniform surface finish, thus reducing the rubbing of the surface to a minimum. All steel was incased in a No. 8 gage steel-wire mesh before pouring the concrete. Steel mesh was also used in the concrete as a reinforcement in the walls between lattice girders.

Many sections of the forms were used a number of times, due to the symmetry of the structure, and this constituted a considerable saving in the material and labor. Nevertheless, as may be seen from the cost table, the cost of the formwork was high as compared with the unit price of concrete.

Mixing and Placing of Concrete

A wooden elevator tower 110 ft. high, made up of verticals of 4x4 in. and 2x6 in. single bracing, with a system of suspended chutes from a movable skip on the tower, adjusted to chuting conveniently different sections of the structure, was used to distribute the concrete. An electrically driven Ransome concrete mixer of ½-yd. capacity was employed, and a gravity hopper delivered to and emptied the concrete in the chutes. On account of the 120-ft. roadway crossing at the center of the site of the station it was necessary to move the tower and mixer to the north end of the station after the south end was completed.

The concreting operation required one hoist runner, one man for emptying the mixer, throwing in the cement and directing the gang at the mixer, three men to handle the sand and stone, one man on the tower to regulate the flow of the mix, and one man at the end of the chute to distribute the concrete. The contract called for a 1:2:4 mix, with broken stone graded to ¾-in. size. A somewhat richer mix was used with enough water in it to compensate for the difficulty and in some cases the utter impossibility of spading the concrete. Notwithstanding this difficulty in spading, the concrete was of a uniform dark appearance with no honeycombing visible and a rich cement layer at the surface, which fact was disclosed when the bush hammer was applied. There were no signs of separation of the ingredients of the mix, although the drop of the concrete from the hopper through the chutes was about 100 ft. in some cases and the mix had water in abundance.

have slotted holes in the bottom flange to facilitate the movement of the platform and wall in one mass. At the surface the layers of cork were finished off with a neat cement wash, and no visible discord in appearance could be detected.

In what way this expansion joint will prevent the scaling off of the concrete and the possible cracks in the concrete, due to the vibrations of the structure in the case of the train impact, will be shown in the future. In connection with this it may be said that the writer has observed the following: The impact of the crane erecting steel about 2,000 ft. away from the station has set the entire steel framework of the station vibrating so as to be noticeable by the writer (standing on a cross-girder) and has thrown out the bubble of the level about six divisions on the tube. This was observed after all the steel was riveted and just before the concrete operations began. It is certain that the weight of the concrete

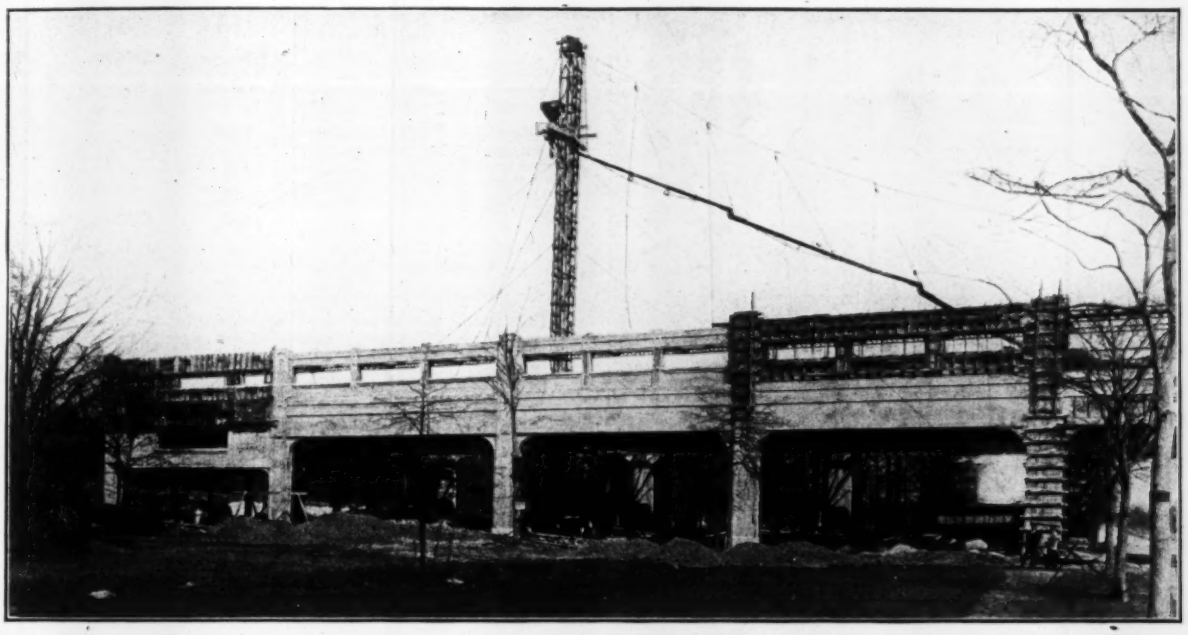


FIG. 3. METHOD OF PLACING CONCRETE IN ELEVATED STATION

It may be of interest to mention that the wooden tower resisted very well a gale of 90 mi. per hr.

Expansion Joints of Special Design

An interesting and quite unusual feature was the method of providing expansion joints for the walls. The structure has four expansion joints located at the same bents as the expansion joints for the track girders. Layers of cork board were used as a separator to keep the concrete apart from the face of the steel lattice column, to provide enough play for the expansion of the walls and at the same time remedy the defect in the outer appearance of the joint. The layers of cork were 1¼ in. thick at the longitudinal expansion joint and ½ in. thick between the bracket and the bottom face of the wall.

The detail of the expansion joint provides a 2x8-in. bearing plate under the wall, resting upon a 4-in. half-round for a smooth longitudinal movement, the shear being cared for by the bracket. The platform girders will materially lessen the vibrations, and the expansion joints will undoubtedly take up some of the vibration. Nevertheless, it may not prove sufficient entirely to prevent cracks and even scaling off some of the concrete.

Waterproofing the Track Floors

The track floors over the mezzanines at each end of the station were waterproofed, the membrane method being used. The process consists of covering the concrete with three plies of a coal-tar saturated burlap in the following manner: The surface of the concrete, when dry, was smoothed off and mopped with asphalt pitch, heated to about 300° F. A ply of burlap was then put on and pressed down carefully upon the mopped surface into the still hot pitch to assure a bond between the ply and the concrete. The surface of the layer was again mopped and a second ply adjusted. This ply was mopped and a third ply put in place and carefully pressed down. The joints overlap each other by a minimum of 1½ ft., care being taken to break joints.

A surface coat of concrete 3 in. thick was then applied and a slope given to each section of the floor, draining through pipes provided in the steel to each end of the floor and thence to the sewer. As a precaution against the water finding its way along the webs of the girders, a 2-in. recess sloping down to the waterproofing fabric was provided and filled with an approved elastic cement compound. This patented compound, costing about $19 for a 129-gal. barrel, is a dark-colored elastic mastic that never hardens. Its function is to fill up the spaces provided between the concrete and steel and eventually fill up all voids in the joints that may occur when the vibrations of the steel will tend to break off the bond between the girders and the concrete, thereby making the point waterproof.

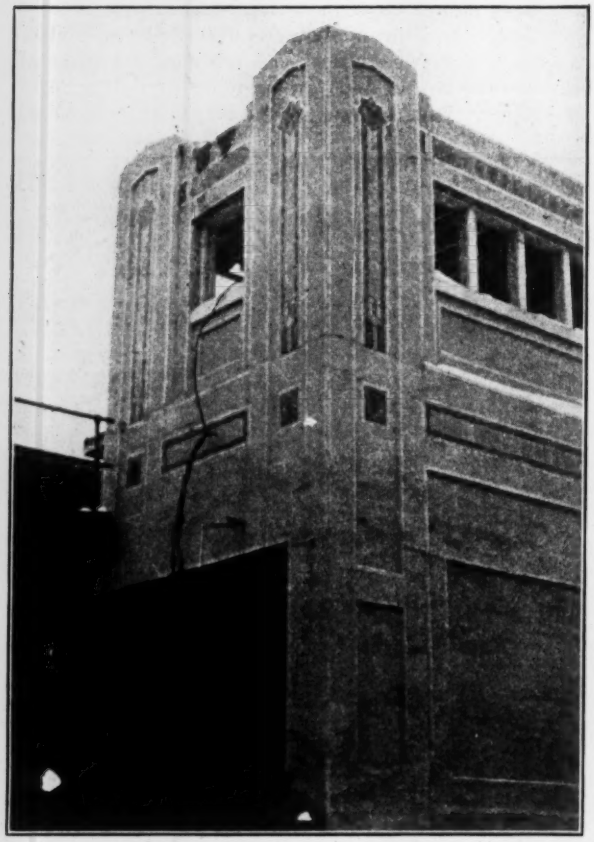


FIG. 4. NEAR VIEW OF CORNER OF STATION, SHOWING ARCHITECTURAL DETAIL

Borders 1½ or 2 in. wide around the windows and panels will be cement washed and rubbed, and the rest of the surface with the exception of the recesses for the inlaid tile will be of a natural concrete finish and tooled, a bush hammer being used. The inlaid colored tile will form diamond and square combinations that will give a pleasant appearance and at the same time break the tendency to expand in one direction. A well-made, wet-pressed, nonerazing tile will be used. This decoration will somewhat soften the austere appearance of the structure.

Costs of Applying Concrete Incasement

The main feature of interest is the cost of concrete work on this station, and especially the cost of the formwork. Although the work was performed quite efficiently, which is the observation of the writer, the item of the formwork was nevertheless $7.73 per cu.yd. of concrete placed, which may be reduced about 47c., allowing for the value of the lumber available for further use after the completion of this job, making the net cost of the formwork $7.26 per cu.yd.

COST OF PLACING CONCRETE ON STEEL FRAME

Total Amount of Concrete Placed = 1,690 Cu.Yd.

				Total	Cost per Cu.Yd.
Lumber, nails, tie-rods, wiring and cost of building and moving the tower				$3,000.00	$1.78
Formwork					
Class of Work	Rate of Pay, per Diem	Number of Days Worked	Total		
Foreman	$5.00	196	$980.00		
Carpenter	4.50	1,520	6,840.00		
Carpenter's helper	2.50	895	2,237.50		
			$10,057.50	10,057.50	5.95
Overhead Charges					
Superintendent	$6.00	92	$552.00		
Timekeeper	2.00	160	320.00		
Watchman	1.50	225	337.50		
			$1,209.50	1,209.50	.71
Concrete Work					
Laborers	$1.50	1,280	$1,920.00		
Hoist runner	4.75	54	256.50		
Mason	4.00	85	340.00		
Mason's helper	2.50	16	40.00		
			$2,556.50	2,556.50	1.52
Concrete Materials					
Material	Unit Cost	Total Materials Used	Total		
Cement, per bbl.	$1.35	2,390	$3,226.50		
Sand, per cu.yd.	.65	435	282.75		
¾-in. stone, per cu.yd.	.90	870	783.00		
			$4,292.25	4,292.25	2.54
Grand total				$21,115.75	$12.50
Appraisal of lumber used for another construction job				800.00	.47
Net total				$20,315.75	$12.03

The price per cubic yard of concrete, as shown in the accompanying table, is $12.03. To this should be added 10% profit for the contractor and 5% insurance charges, or $1.80, making a total of $13.83. The price asked by the contractor in his successful bid was $11 per cu.yd., thereby resulting in a net loss to the contractor of $2.83 × 1,690 = $4,782.70. This loss was successfully offset, however, by the prices obtained for the steel, wire-mesh, surface-finish and tile work.

It may be of interest to mention that this contract for the concrete was sublet at the beginning of the undertaking for a price of $8 per cu.yd., for constructing the forms, supplying the cement, sand and stone and placing the concrete, all lumber, wires, nails, bolts, etc., and overhead charges being supplied by the Oscar Daniels Co. The subcontractor continued operations from June 28, 1915, to Oct. 12, 1915, after which date he was forced to retire, realizing a big loss of money due to the incompleted work. At the failure of the subcontractor to fulfill the requirements, the Oscar Daniels Co. continued and successfully completed the entire project.

The cost data were compiled from strict accounts of all the labor performed and materials supplied from the day of the beginning of the work to the day it was completed.

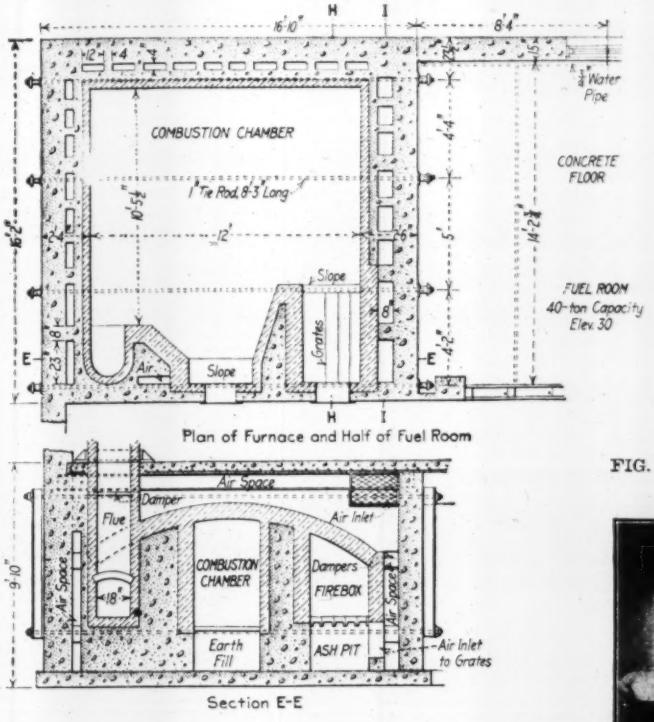
The work was done under the supervision of the Public Service Commission, George Paaswell, assistant engineer in charge. The writer was in close connection with the engineering and supervision of construction.

The Cost of Freight Cartage in city streets is to be the subject of investigation by the United States Census Bureau. It will be the object of the inquiry to determine what part the cost of carting goods to and from the railway station plays in the high cost of living.

Dutch-Oven Garbage Incinerator, Mason City, Iowa

By Robert F. Gayton*

A garbage and refuse incinerator consisting of two ten-ton dutch-oven-type units was recently put in use at Mason City, Iowa. Each unit is 12x12 ft. in plan by 4½ ft. high, inside. Figs. 1 and 2 show the details and general arrangement of the furnaces. The tops of the furnace form the floor of the second story. The second story covers but half the building and is the driveway and charging space. The building is 32x50 ft. in plan, built of tile, with a pebble-dash outside finish. The roof has a steel frame and is covered with red asbestos shingles.



The furnaces were built under a patent (U. S. No. 1,107,615) held by Jones Bros., Macon, Ga. The flues are arranged to give a preheated air supply, so that 60 lb. of the average mine-run bituminous coal is expected to reduce one ton of city-run garbage. This city-run garbage consists of kitchen wastes, commercial wastes—including tin cans and bottles—all carcasses, and night soil. The process is odorless at 1,000° F. and higher. The furnaces reach this temperature one-half hour to an hour after the fire is started. The process is also smokeless. The ash is approximately 10% of the garbage bulk and from 5 to 7% of its weight. The smokestack, lined with firebrick, is 18 in. in diameter and 40 ft. high.

When the test of the crematory was made, several tons more garbage was consumed than the guarantee called for.

The refuse-collecting wagons dump directly into the furnace (Fig. 2). By the use of the underslung wagon a load is dumped in three minutes. It is rarely necessary to raise the temperature in the furnaces higher than 2,200° F.

The furnaces are in continual operation day and night. Each two weeks it is essential to completely burn out a furnace to clean out the flues, which is a simple matter.

The cost of the incinerator complete was $12,000. As a matter of interest it may be stated that these burning units can be installed singly, in pairs or in any number necessary in accordance with the quantity of waste material to be consumed.

In connection with the incinerating plant there will be completed in February, 1917, sewage-treatment works consisting of Imhoff tanks and sprinkling filters. The cost of the entire system, including the incinerator, will be approximately $160,000.

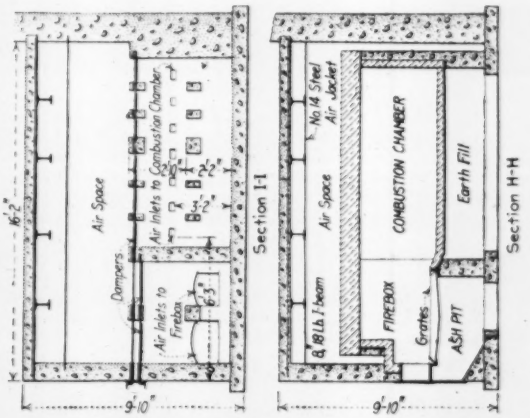


FIG. 1. PLAN AND SECTIONS OF GARBAGE INCINERATOR UNIT, MASON CITY, IOWA

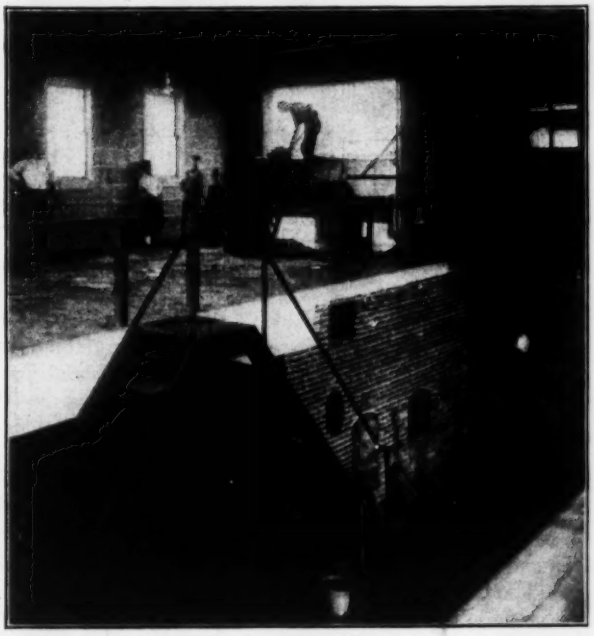


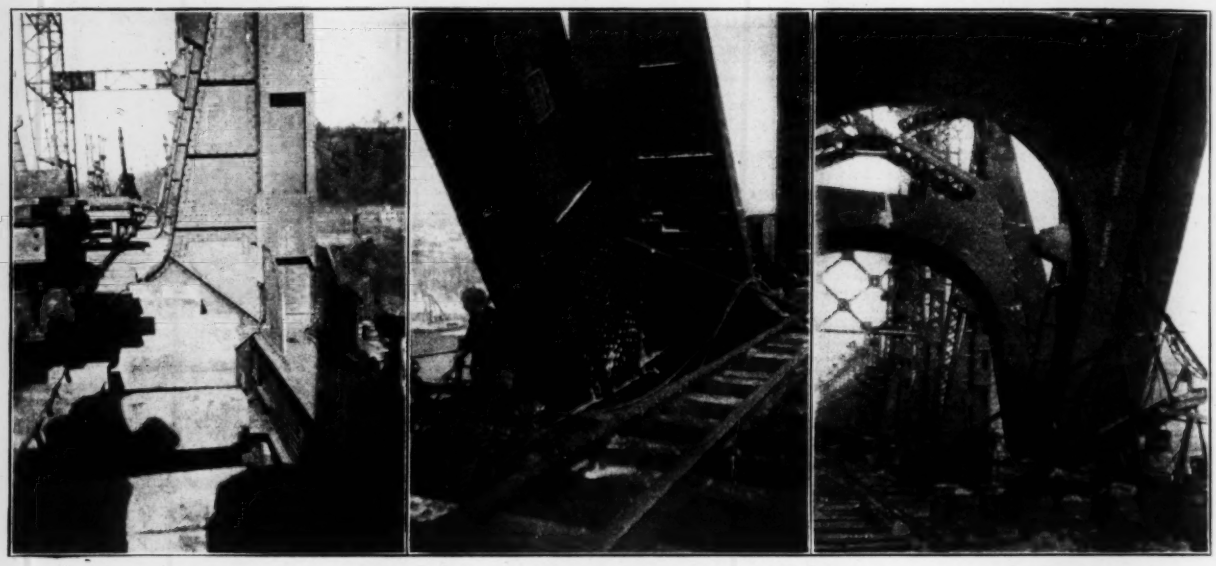
FIG. 2. CHARGING AND STOKING FLOORS, MASON CITY GARBAGE INCINERATOR

*Assistant City Engineer in Charge of Construction, Mason City, Iowa.

The Cleveland Municipal Electric Station, which was completed in July, 1915, with a capacity of 20,000 kw., is to be increased 50% in capacity by the purchase of a 10,000-kw. generator, to be delivered in thirteen months. The price of the generator is $112,500, and it is to be paid for from the plant's earnings.

Sciotoville Bridge Erection Is Well Started

An unusual bridge-erection problem is being handled in the construction of the Sciotoville Bridge over the Ohio River for the Chesapeake & Ohio Northern R.R., which has been designed and is being executed under the direction of Gustav Lindenthal, Consulting Engineer, the McClintic-Marshall Co., of Pittsburgh, being the contractors. The unusual feature is the bending of the truss members during erection in such direction and amount as to bring all members into precisely straight condition when the completed bridge is loaded to half its live-load capacity. The object is to eliminate secondary stresses, not only in the chords but also in the web members.



FIGS. 1 TO 3. ERECTION VIEWS OF THE SCIOTOVILLE BRIDGE

Fig. 1—Floor-beams and lower post sections set; gantry being raised for truss erection, Nov. 1. Fig. 2—Main panel point (L 16). Fig. 3—Portals over the center pier.

Something similar was done in the new Quebec Bridge, but in simpler manner on account of the numerous pin connections. The Sciotoville Bridge has riveted joints throughout.

The erection program is based on full and precise calculations covering the deflections in every stage of erection, the stresses in all members, the influence of raising or lowering any point of support, the angular relations of the triangle members, and the forces that must be applied to bring the members together.

The bridge comprises two 775-ft. spans continuous over the middle pier—a structure virtually of new type for America, and the largest structure of its type ever attempted. It was shown by general drawings in *Engineering News* of July 8, 1915, p. 64. The original feature of U-shaped floor-beams, whose upward-extending legs are held at the top by the sway-bracing struts thus securing both stiffness and reduced floor-beam moment, was there shown by a drawing. These floor-beams in erected position are seen in the view, Fig. 1, herewith. They are field-spliced a short distance above floor level, the upper parts being built integral with the posts. This splice was drilled to templet in the shop, except for the splice cover on the inner flange, which was drilled in the field.

The views show the Ohio span, which is erected on falsework; the Kentucky span is to be cantilevered out from it for a length of 465 ft., will there land on a steel temporary bent, and 155 ft. farther will land on a second bent, 155 ft. north of the Kentucky end pier. On account of the deflections of the trusses as the erection progresses, and the necessity of avoiding both undue stress in the truss members and excessive concentrations of load on the falsework posts, the truss will be jacked up several inches at these temporary bents as soon as it reaches them.

Similar jacking is done on the Ohio span previously, the truss being jacked up successively at three points, as it reaches them, in order to free the falsework on which it rested up to that time and thereby allow this falsework to be pulled, which will eliminate a large part of the danger from drift in an unexpected flood.

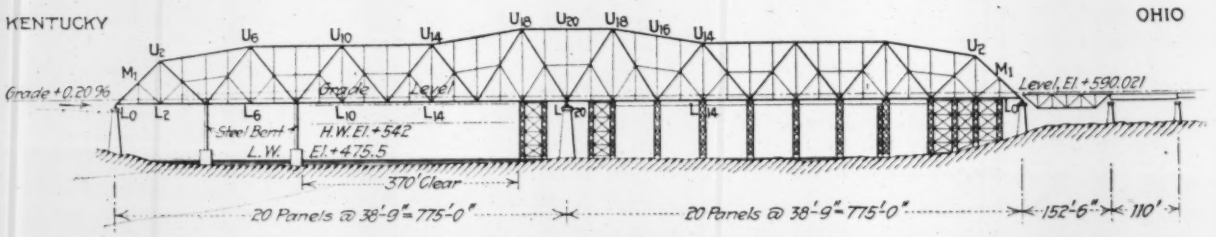


FIG. 4. SKETCH OF ERECTION FALSEWORK

The diagram Fig. 4 is reproduced from an erection sketch. The description of the erection procedure is based in part on a statement prepared by Paul L. Wolfel, Chief Engineer of the McClintic-Marshall Co.

Timber falsework is used to support the floor at every second panel point of the Ohio span, and plate girders from the approaches are used to span between them to carry the intermediate joints. At every second falsework bent (that is, every main triangle point) steel columns are built in, on separate footings, to carry the truss load. Screw-wedge jacks of 250 tons capacity on the timber bents (Fig. 7) allow for adjustment of height. Pin bearings are placed on the steel columns, allowing space for jacking up the truss by hydraulic jacks (Fig. 6).

The Ohio-span floor was first laid, by gantry traveler proceeding from L 0 Ohio, to L 20, the center pier, and on to L 18 Kentucky. Then working back, the traveler placed the lower chord. Immediately the splices of the chord were riveted. Then the chord was jacked and lowered at its various bearings, to deflect it into a curve of such shape as to allow the web-member connections to be made most easily. This curve is at its maximum about 1½ in. above the true level line, and at L 0 Ohio is about 8 in. below level.

The gantry being now built to the height necessary for truss erection, and the lower sections of the posts placed and spliced to the floor beams and chord, the work on the trusses was started at the great triangle over the center pier, and is progressing toward the Ohio bank. To erect the cantilever work of the Kentucky span, a creeper traveler was erected on the top chord of the L 20, U 18 O, U 18 K triangle in the latter part of December.

In the erection of the web members and top chord, none of the joints will go together without forcing, as they were laid out to fit the true truss diagram when stressed under full dead- and half live-load (3,000 lb. per lin.ft. per truss, full live-load being equivalent to about 6,000 lb. per ft. per truss). The web diagonals in general require to be bent by upward pulling with the tackle of the traveler, until drift pins can be entered in the joint, and then the connection must be forced into full matching position by drifting of a kind that will tend to put a slight reverse bend into the member.

To connect the posts, large vertical forces are required, for which purposes a special jacking rig has been built, to be clamped around the post near the top.

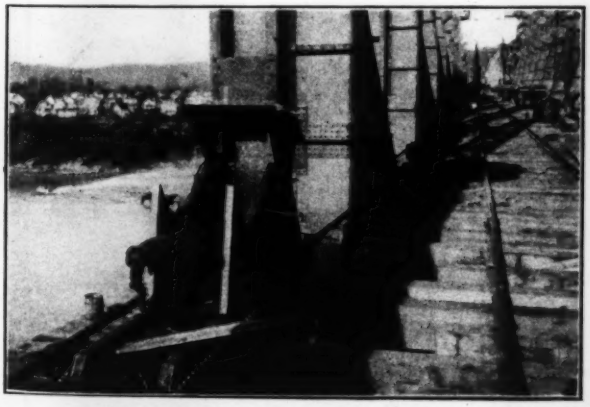
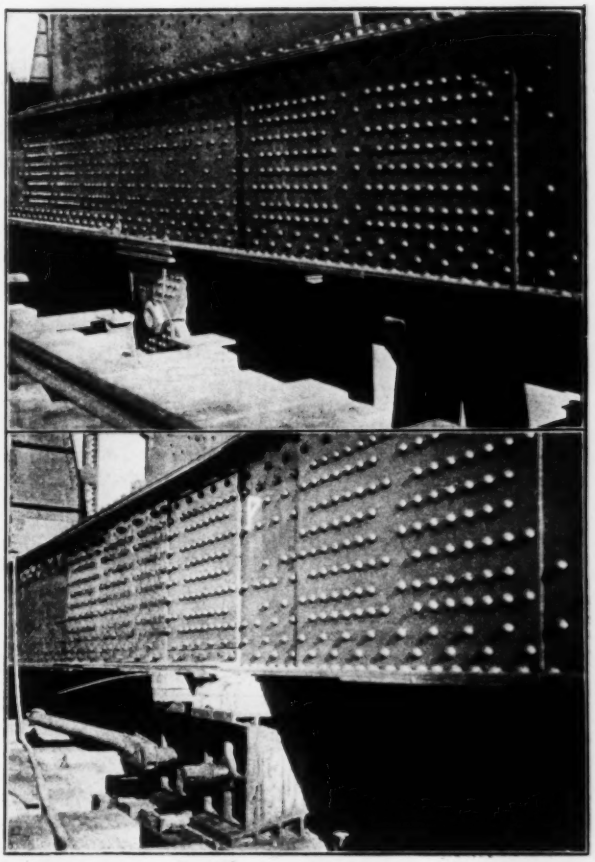


FIG. 5. THE U-FLOOR BEAMS SEEN FROM THE CENTER PIER; GUSSETS OF L 20 AT LEFT



FIGS. 6 AND 7. PIER SUPPORT AT MAIN PANEL POINT AND WEDGE JACK AT INTERMEDIATE PANEL POINT

A steel post is built in the falsework under each truss at the four main panel points

The Ohio span will be jacked up at L 12, L 8 and L 4 successively, to free the falsework. The jacking loads range to 400,000 lb.; the greatest load on the falsework columns will be about 1,300,000 lb. Finally, the Ohio end will still be 8.1 in. low and will have to be jacked up with 1,926,000 lb. to proper level, which will allow the 16-in. end rollers to be inserted and will free the last of the falsework columns.

Hydraulic jacks of 200 and 500 tons capacity are provided for the jacking; they will utilize pressures up to 4,000 lb. per sq.in.

Jacking the Kentucky span on the bents at L 8 and L 4 will require somewhat larger forces than above noted, and the final jacking at L 0 Kentucky will require a force of 2,337,000 lb. and a 16-in. raise.

At the end of December the trusses are erected from L 18 Kentucky to L 16 Ohio. The creeper traveler is in place on the Kentucky top chord and is taking down the falsework at 18 and 19 Kentucky. The gantry is proceeding with erection of the Ohio span. When L 12 Ohio is reached, the truss will be jacked. Shortage of labor has caused some delay, especially in riveting. The chord splices and large connections have 1¼-in. rivets, which demand skilled riveters.

R. T. Robinson is Resident Engineer for the consulting engineer. A. Toohey is superintendent in charge of erection for the contractor.

Notes from Field and Office

Diagram for finding column sections—A shaft-sinking record—Slide-rule for regearing water meters—Concrete pond curb good after 10 years—Small shield makes good record

Column Sections Quickly Found by Novel Diagram

By H. G. Nevitt*

The column formula of the American Railway Engineering Association is

$$p = 16{,}000 - 70\frac{l}{r}$$

where p is the unit load on the column, in pounds per square inch; l is the unsupported length of the member, in inches; r is the radius of gyration of the section, in inches. If P be the total load in thousands of pounds and A the sectional area in square inches, then

$$P = \frac{p}{1{,}000}A = \left(16{,}000 - 70\frac{l}{r}\right)\frac{A}{1{,}000}$$

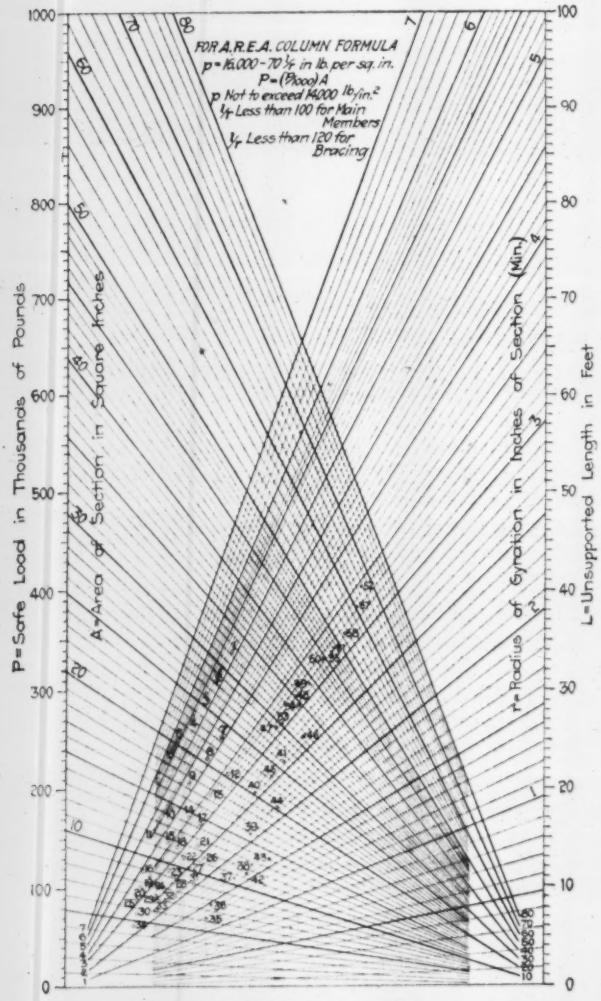
Of the four quantities to be considered, P and l are determined from the conditions of design. It then remains to choose a section for which r and A exactly satisfy the above equation, or in practice give a loading $\frac{P}{A}$ equal to or less than the allowable p. In addition, any other requirements demanded by the specifications or conditions of design, such as the limiting value of the ratio $\frac{l}{r}$, must be satisfied.

In the diagram, scales for P and L (L is length in feet, l is length in inches) are laid off on parallel vertical lines. Between these is a system of coördinate lines, radiating from the two initial points, representing the quantities A and r respectively. Hence any point between the scales for P and L has definite coördinates, or corresponds to a set of values for A and r. The property of the diagram is that a straight line between the points on the scales corresponding to P and L, respectively, passes through all the points whose values of A and r satisfy the equation with the value of P and L chosen. Hence, knowing P and l, a value of A can be assumed and the necessary value r read, or vice versa.

Evidently, any column section can be represented by a point, for it has a particular A and r. And if this point lies on the line through P and L, it gives the intensity of stress allowed by the column formula. If above it, the intensity will evidently be less than the allowable, and greater if below. Hence in brief:

Directions—Lay a straight-edge between the points on the scales for P and L. If the point whose A and r are

*Engineering Department of H. L. Doherty & Co., 60 Wall St., New York City.


FOR A.R.E.A. COLUMN FORMULA
p = 16,000 - 70 l/r in lb. per sq. in.
P = (P/1000) A
p Not to exceed 14,000 lb/in.²
l/r Less than 100 for Main Members
l/r Less than 120 for Bracing
P = Safe Load in Thousands of Pounds
A = Area of Section in Square Inches
r = Radius of Gyration in Inches of Section (Min.)
L = Unsupported Length in Feet

A NOVEL STRAIGHT-LINE DIAGRAM FOR DETERMINING COLUMN SECTIONS UNDER A. R. E. A. FORMULA

LATTICED CHANNEL COLUMNS

No.	Depth, In.	Wt. per Ft., Lb.	B. to B., In.	No.	Depth, In.	Wt. per Ft., Lb.	B. to B., In.	No.	Depth, In.	Wt. per Ft., Lb.	B. to B., In.
1	15	55		13		30		24		13.75	
2		50		14		25	7	25		11.25	5
3		45	9½	15		20		26	7	19.75	
4		40		16		15	7½	27		17.25	
5		35		17	9	25		28		14.75	4
6		33	10	18		20	5½	29		12.25	
7	12	40		19		15		30		9.75	4½
8		35		20		13.25	6	31	6	15.5	
9		30	7½	21	8	21.25		32		13	3½
10		25		22		18.75	4½	33		10.5	
11		20.5	8	23		16.25		34		8.0	4
12	10	35									

PLATE AND ANGLE COLUMNS

No.	Web, In.	4LS	No.	Web, In.	4LS	2PLS
35	6x¼	3 x2x¼	47	12x¼	6x4x⅜	14x ¼
36	8x¼	3½x2½x¼	48	12x¼	6x4x7/16	14x ¼
37	8x7/16	4 x3x7/16	49	12x¼	6x4x½	14x ¼
38	8x⅜	4 x3x⅜	50	12x¼	6x4x9/16	14x ¼
39	10x⅜	5 x3½x⅜	51	12x¼	6x4x⅝	14x ⅜
40	10x⅜	6 x4x7/16	52	12x¼	6x4x⅝	14x1 ⅜
41	10x⅜	6 x4x7/16	53	14x⅜	6x4x⅝	14x ⅜
42	12x⅜	4 x3x7/16	54	14x⅜	6x4x⅝	14x ⅜
43	12x⅜	4 x3x⅜	55	14x⅜	6x4x⅝	14x ⅜
44	12x⅜	5 x3½x⅜	56	14x⅜	6x4x⅝	14x ⅜
45	12x⅜	6 x4x½	57	14x⅜	6x4x⅝	14x1
46	12x⅜	6 x4x9/16				

those for the section considered lies on or above the line, the intensity of stress will be equal to or less than that allowed by the formula, and the section may be used if desired.

Example—P = 170,000 lb., L = 30 ft. Then any one of sections 9, 12 and 41 will be safe, and we use 9, which is two 12-in. by 30-lb. channels latticed, as having the smallest area.

Combined bending and compression may be treated in exactly the same manner if we use instead of P the quantity $\left(P + M\frac{n}{r^2}\right)$, where M is the bending moment in inch-pounds and n is the distance to the extreme fiber of the section in inches. This, of course, necessitates a solution by trial, which is without difficulty.

Method of Plotting Diagram

The following is the method of plotting a similar diagram for any column formula of the straight-line type. The theory is omitted for the sake of brevity, but is comparatively simple.

Assume the formula adopted as

$$p = X - Y\frac{l}{r}$$

the quantities being defined as before and X and Y being the coefficients in the formula.

Then: (1) Lay off on parallel lines the quantities P and L to any desired scales; (2) construct the coördinate lines for A so that they pass through the zero point of the L scale and intersect the P scale at the points $P = AX$; (3) construct the coördinate lines for r so that they pass through the zero point of the P scale and intersect the l scale at the values $l = \frac{X}{Y}$.

For convenience, on the A and r coördinates plot known sections for reference, although A and r may be read directly from the diagram and a suitable section selected if desired.

A disadvantage of the diagram is that a slight inaccuracy is introduced by unequal shrinkage of the paper. This is usually negligible. Another disadvantage is the limitation of accuracy by size. This objection is small here and can be entirely overcome by the use of different diagrams for different classes of work, as building columns and bridge upper chords.

Advantages of the diagram are: Speed of computation, combined with all accuracy usually necessary; compactness, as compared to tables; ease and simplicity of construction and use. Graphical comparison of the weight-efficiency of different sections, when the points are plotted on the diagram. For example, the latticed-channel columns have their metal better distributed than the plate-and-angle columns.

Shaft Sunk 252 Ft. in 31 Days

What is claimed to be a world's record in shaft-sinking was made by the Crown Mines in its No. 14 shaft, located in the Bovysers Reserve on the South African Rand. According to A. Cooper Key in the *Engineering and Mining Journal*, in the 30 days of September and the first day of October, this shaft was sunk 246 ft. and 6 ft. respectively, totaling 252 ft. It is circular, 21 ft. 6 in. diameter before bricking, 20 ft. inside lining.

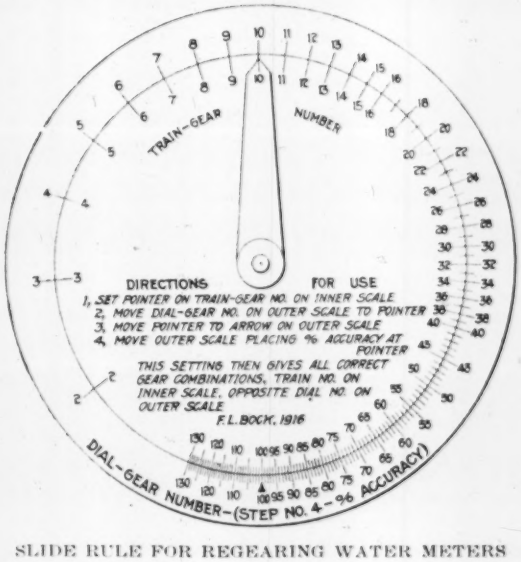
The sinking was done with the aid of 30 natives, working 90 shifts. Five Ingersoll-Rand machines were used per shift. To break the 9,120 tons of rock excavated, 67 cases of gelatin were consumed. This rock was hoisted in a 3-ton bucket, the hoisting being done by one steam engine connected with two boilers.

The previous sinking record was made by the Modder Deep Levels in April, 1912, when 231 ft. was sunk in one of the twin shafts. This shaft had three compartments, two measuring 5x6 ft. and one 6x6 ft. The Modder Deep beat the record created a month earlier in the southeast shaft of the Modder Government Gold Mining Areas by the narrow margin of a foot. This, in turn, was an advance from the 213 ft. of the Modder Deep constituting the previous record. Previously the best record was that of the Brakpan Mines, made as far back as July, 1907.

The 1912 records, now surpassed, were made at a depth of 2,000 to 2,500 ft., whereas the total depth of the Crown Mines new shaft is about 1,000 ft.

Slide Rule for Regearing Water Meters

A slide rule devised for the use of nontechnical employees of a water-works in computing combinations of change gears was described and pictured in *Engineering News*, Oct. 26, 1916, p. 800. The same scheme has been employed by the originator, F. L. Bock, Assistant Engineer, Distribution Section, St. Louis Water Works, in a circular rule that is more compact, more easily made and more convenient to use. As shown in the accompanying sketch, it consists of two concentric disks, the outer edge of the inner one and the inner edge of the outer one cutting the numbered radial lines. A rotating pointer takes the place of the pointer or hair-line rider of the straight rule. The new rule is used as directed in the sketch; it is first required to know the number of teeth on the old gears and the accuracy of the meter with them.


TRAIN-GEAR NUMBER
DIRECTIONS FOR USE
1, SET POINTER ON TRAIN-GEAR NO. ON INNER SCALE
2, MOVE DIAL-GEAR NO. ON OUTER SCALE TO POINTER
3, MOVE POINTER TO ARROW ON OUTER SCALE
4, MOVE OUTER SCALE PLACING % ACCURACY AT POINTER
THIS SETTING THEN GIVES ALL CORRECT GEAR COMBINATIONS, TRAIN NO. ON INNER SCALE, OPPOSITE DIAL NO. ON OUTER SCALE
F.L. BOCK, 1916
DIAL-GEAR NUMBER-(STEP NO. 4-% ACCURACY)

SLIDE RULE FOR REGEARING WATER METERS

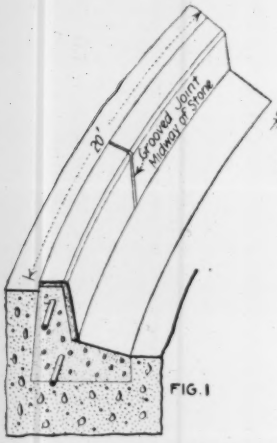
Concrete Curb of Park Ponds Uninjured After 10 Years

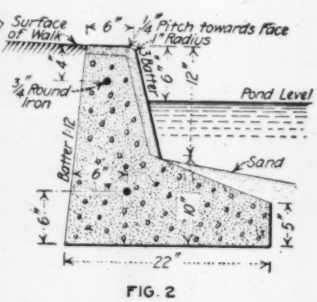
By Howard Lathrop*

Wading and skating ponds were built in the public parks of Fall River, Mass., varying from about ¼ to 1 acre of water surface. The margins were not regular curves connected by tangents, but a series of irregular curves with no straight lines whatever. Above the water line was a strip of sand with about a 1 in 10 pitch and 2 ft. wide, and surrounding this strip of sand was a macadam walk, both extending entirely around the ponds.

The wave action, when a strong wind blew, caused the sand and then gradually the walk to wash down and scatter, resulting in a very ragged margin.

Some form of curbing was necessary to confine the water to its original limits. To cut a stone curb to fit the series of irregular curves and set it to withstand frost action and ice pressure would have entailed considerable cost, inasmuch as each stone would have to be cut on a curve to fit its special position, and probably no two stones would be alike. Consequently, a concrete curb was decided upon as the cheapest and simplest. The left-hand sketch shows the curb in perspective, while the right-hand shows the cross-section adopted.





CROSS-SECTIONS OF CONCRETE CURB

The back and face of the curb slope—the back in order to present less resistance to frost action, and the face to allow the ice to slide readily upward when expanding and not force the curb out of alignment. The writer has seen the ice forced up and over the top of the curb, due probably in a great measure to the heavy vibration caused by hundreds of skaters gliding over ice having less than 1½ ft. of water under it. A foundation of cinders was made because in long cold spells the water, by forming into ice and by seepage, would likely be at a lower elevation than the curb. As cinders were plentiful and could be had for the hauling they were also placed as a backing against the curb.

The curb, as shown, served to retain the surrounding walks some 12 in. higher than the bottom of the pond. The curb was divided into 20-ft. lengths by expansion joints and laid in alternate blocks to facilitate their construction, and to provide for the placing of dowels.

*Superintendent, Fall River (Mass.) Park Department.

The forms were of the usual construction, except the form to hold up the face of the curb, which was No. 1 planed spruce, greased. They were erected in a continuous run, kept about a day ahead of the concrete and adjusted finally for line and grade just previous to placing the concrete. The expansion joints were formed by placing one thickness of three-ply tarred paper against the end of the finished section before constructing the adjoining length. To prevent the curb from being forced out of alignment two ¾-in. diameter wrought-iron dowels 18 in. long were placed at each joint half their length in each section. These dowels were wrapped in paper and carefully placed to prevent any bond between the dowel and the concrete. By this means contraction and expansion were provided for, but forces tending to throw the sections out of alignment were prevented.

The top of the curb and its face were troweled and finished similarly to ordinary granolithic work. The body of the concrete was a mixture of one part portland cement, three parts coarse sand and five parts crushed granite of No. 2 size (¾ to 1½ in.). The mix was placed rather wet in 6-in. layers, to save excessive tamping against the forms. The concrete was spaded against the back and toe of the curb to insure as smooth a surface as possible. The surface layer on the face of the curb was placed by plastering against the forms ahead of the concrete (keeping the plastering just above the concrete), insuring a complete bond, but giving a varying thickness of the surface layer from ¼ to 1 in. At no place was the body of the concrete allowed to approach the outer surface.

The concrete was left 1 in. low on the top of the curb and was immediately covered to the top of the form with a 1:2 surface layer in which the aggregate was coarse sand mixed half-and-half with fine crusher dust and colored with lampblack. The face form was carefully removed, and the face and top of the curb were floated and troweled to a hard smooth surface. The jointer was run at each expansion joint and in the middle of each section, giving the appearance of 10-ft. stones. A curved edger was run over the junction of the top and face of the curb.

Two ponds were thus encircled at a total cost of $1.69 per ft., there being 1,621 lin.ft. This cost includes every item connected with the work, which was done in 1906 and, except for the natural 10 years' wear, is in good condition.

Small Sewer Tunnel Makes Rapid Progress by Shield Work

In tunneling for a 48-in. concrete circular sewer in West Water St., northward from Florida St., Milwaukee, Wis., highly satisfactory results have been obtained on one section by using a shield. The soil is fairly stable and compressed air is not required; in fact an unbreasted face can be maintained most of the time. In spite of this condition the contractor's superintendent was convinced that he could do better with a shield than with square timbering as employed in other sections of the same work. The shield was built, and experience has justified its use. The rate of advance obtained is two or three times as great as with timbered excavation. The maximum has been 37 ft. in two 10-hr. shifts.

The shield is 7 ft. in diameter and about 9 ft. long. Its shell plate is stiffened by a circular girder inside in the usual way, and extends forward to a bracketed cutting edge of equal length all around (no hood). The tail is long enough to permit of laying up four or five rings of the wood segmental lining that is used to hold the excavation before concreting. There are seven jacks, but usually only a few of them are used. The heading foremen have acquired skill in selecting and controlling the jacks so as to guide the shield in alignment or grade as local conditions require. At the start of the work they were without shield experience and the path of the shield was rather sinuous, diverging by several inches to one side and to the other. After a few hundred feet, however, they became skilled enough to secure a very satisfactory straight path and could guide the shield at will.

The lining consists of 4x5-in. wooden cants or segments, with the 5-in. side radial. The tunnel wall is to be 9 in. thick (net), of unreinforced concrete. This thickness gives 4 in. leeway on either side, to allow for divergences of the timbered excavation from true line and grade. The concrete is, of course, carried tight up against the timbering.

The soil is a sandy clay occasionally somewhat fibrous, locally called marsh clay, overlying a peaty stratum over sand. The tunnel is 15 to 25 ft. below street surface. The section on which the shield is employed is being constructed by John F. Casey Co.

The 5-in. thickness of wood cant lining appears to be ample to resist the jack pressure required in forcing the shield forward. The lining compresses visibly for some 30 ft. back of the shield, and in some instances evidences of compression have been noticed as far as 100 ft. back. No ill results of this compression have been observed, however. Concrete will not be placed until the entire length of the shield section is completed.

The shield is advanced about 18 in. at each shove, the cutting edge being forced into the soil after the middle portion of the face has been excavated ahead. The shove brings down about half the amount of material displaced at each advance, and this requires only to be shoveled out. Subsequent excavation of the central part of the face is done by mattock.

The speed of advance of the method is essentially due to the assistance to excavation by the shove of the shield and eliminating the delay due to timbering, as the segments are easily and quickly placed.

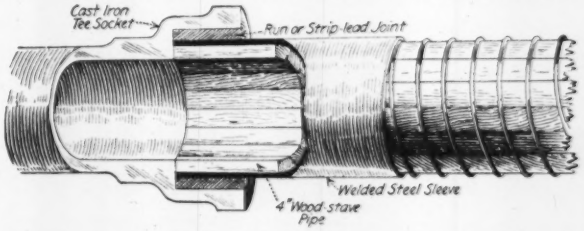
This sewer is part of the elaborate interceptor system now being built under the Milwaukee Sewerage Commission, T. Chalkley Hatton, Chief Engineer.

FINE AGGREGATE

Geographic Positions of Familiar Points may be obtained from bulletins published by the United States Geological Survey. These give latitudes and longitudes of marks at crossroads, township corners, etc., throughout given areas. One for the middle northwestern states has just been issued.

Accurate Triangulation for a Large Bridge—To locate the piers of the Metropolis Bridge over the Ohio River on the new Paducah & Illinois R.R. line of the Chicago, Burlington & Quincy Ry., very precise triangulation work was done. The low-water lines of the river are about 2,800 ft. apart, and the entire length of steelwork is 5,480 ft. A base 2,000 ft. long (approximately) was laid out on each bank, with one of its ends on the bridge axis. Measurement of these bases by refined tape methods determined their lengths with a probable error of 1:1½ million and 1:2 million respectively. Then the twelve angles of the quadrilateral were measured with a 10" transit having 8-in. circle. The greatest error of closure was 3". These errors were distributed, and the long sides of the quadrilateral computed. Taking the plan distances between piers and a fixed location for the first pier, the angles to the several pier centers were computed. To locate the piers the best angle from the ends of the base was chosen, and one transit set to this angle while another was lined to range in the bridge axis. No final check on span lengths will be obtained until the bridge superstructure is completed, as the only long tape available for measuring between piers was not standardized. However, the longest span of the bridge, the great 720-ft. channel span on the Kentucky side, joined up with all desired accuracy. W. McCready, Principal Assistant Engineer, carried out the survey work, under C. R. Fickes, Chief Engineer, Paducah & Illinois R.R.

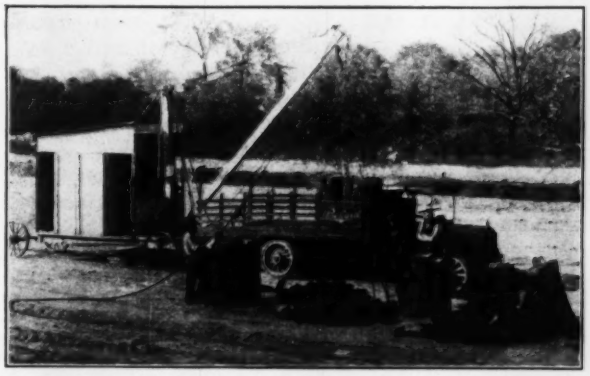
Joints Between Cast-Iron and Wood-Stave Pipes—Where special castings for hydrants, valves and other connections are inserted in wood-stave pipe, necessitating a lead joint between wood and iron, leakage is liable to occur owing to lack of adhesion between wood and lead. The difference in density of the two substances makes the proper calking of



METHOD OF FORMING JOINTS BETWEEN CAST IRON AND WOOD

the lead packing practically impossible. T. Pridham, in the "Commonwealth Engineer" (Australia), shows a method of remedying this defect by providing a joint that can be made fairly water-tight even under high pressure. The accompanying sketch is self-explanatory.

Unloading Paving Blocks with a portable derrick is shown in the accompanying illustration. The blocks are 2x5x12-in. asphalt, furnished and laid by the Asphalt Block Paving Co., Toledo, Ohio, for the paving of Shaker Boulevard between North Moreland and Coventry, suburbs of Cleveland, Ohio. The blocks were unloaded from the railway cars by placing



PORTABLE DERRICK UNLOADING PAVING BLOCKS

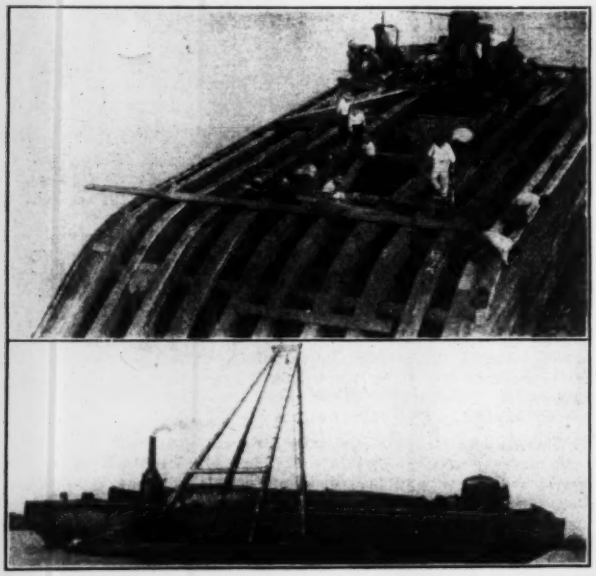
them in cages, like those shown, each cage holding about 100 blocks; these loaded cages were lifted from the cars and placed on motor trucks by a portable derrick. Another derrick of the same type was kept out on the road to unload the truck, as shown in the illustration. The cages were dumped and the blocks neatly piled by a crew of two, between trips of the truck. The derrick is moved from place to place by coupling it to the motor truck.

Mixed Wagons Can Be Started by a Steam Shovel, as is pointed out in Bulletin 18 of the Osgood Co. When wagons become mired in the loosely excavated material, so that it is difficult for the teams to start them, the dipper of the shovel can be placed at the rear end of the wagon and by means of a slow thrust from the crowding or boom engines, or by swinging the shovel, the wagon is forced to travel.

A Loose-Leaf Notebook of tracing cloth is urged by Roy C. Hardman, Box 455, Balboa Heights, C. Z. He says:

Sheets of this sort have a very decided advantage over paper sheets in that copies of notes may be readily obtained by blueprinting, and sketches may be traced from magazines and other outside sources. Matter may also be typewritten on the cloth sheets and clearly blueprinted if a black ribbon is used, and a black carbon copy be made of the reverse side of the cloth by the simple process of reversing the carbon sheet. As tracing cloth can be secured with different rulings, such as cross-section and profile, almost any sort of notes desired by an engineer can be made. An additional advantage is gained over paper inasmuch as the cloth is less liable to tear out at the binding rings of the loose-leaf binder. The cloth will also bear a great deal more wear than paper.

Salvaging a Hopper Scow in the Hawaiian Islands is shown in the upper view. The scow belongs to the Hawaiian Dredging Co., is 94x36 ft. in plan by 13 ft. deep and draws 4 ft. light and 10 ft. loaded, the loading capacity being 300 cu.yd. Some time ago the scow broke loose in a storm and drifted on the rocks. An anchor scow, brought alongside during the work of examination to determine the possibilities of rebuilding added to the damage by colliding with the al-



SCOW BOTTOM UP WITH REPAIRS MADE; RIGHT SIDE UP AND O. K.

ready wrecked scow. The scow had cost $15,000 to build, and it was estimated that the repair charge would amount to $3,000. The actual repair cost reached $6,000. To repair the bottom, the scow was rolled over in the water, it being shown upside-down in the upper view. The lower view shows the hull rightside up again, after repairs had been completed.

Pipes First Are Struts, Then Drains—Rudolph N. Maxwell, 3541 Trimble Ave., Cincinnati, calls attention to a retaining wall of the Pennsylvania R.R. at Torrence Road station, near Cincinnati, in the recent building of which lengths of cast-iron pipe were used first as bracing for the foundation trench and ultimately as weep holes. The excavation, which was some 15 ft. deep, had been very heavily timbered to prevent the hill above from sliding. The crossbraces or struts that ran from front to back, holding the horizontal rangers in position, consisted mainly of 8x8s, but these were placed so closely together that it would have been unwise to leave them in the concrete. Not only would they have considerably reduced the concrete volume of the wall, but they would also have endangered it, owing to the swelling of the wood during concreting. To overcome the difficulty, heavy 2-in. cast-iron pipe was substituted for the wood crossbraces before concreting. The pipe was cut to the proper lengths, and as one pipe was fitted alongside a corresponding wooden brace, the brace was removed. This was continued until all the struts had been taken out and only the pipe remained.

Moving a Town of 51 houses would seem to be a job of some magnitude, but a contract for this very work has been let by the Town of Barfield, Ark., to J. N. Thompson, of New Madrid, according to press dispatches. Barfield is located on the Mississippi River, and caving banks have necessitated the construction of a loop in the levee at this point. The houses are to be moved back out of the loop, and the cost of the moving is estimated at $10,000.

Hillside Asphalt-Block Pavement is now being laid in the same manner that wood-block pavement has been constructed under similar conditions. Between each row of blocks a narrow strip of wood or lath is placed so as to separate the

ASPHALT-BLOCK PAVING ON GRADES

rows by ⅜ to ½ in. The view shows a pavement being put down in this manner on the Albany Post Road in the village of Dobbs Ferry, N. Y. A total of 40,000 sq.yd. was laid on this contract by the Hastings Pavement Co., New York City.

In Patching a Brick Pavement on the Warrensville road, near Cleveland, Ohio, a small Schramm portable air compressor and a pneumatic tool were found useful and economical. The pavement was comparatively new and was damaged, it is claimed, by a heavy motor truck before the foundation concrete had properly set. The edges of the patch were first cut with the pneumatic tool, the ends of the brick be-

AIR TOOL USED FOR MAKING PATCHES IN A BRICK PAVEMENT

ing carefully cleaned of all old mortar. The main part of the patch was then broken up and removed with a sledge hammer and pick. The air tool was also used to clean the brick, which were to be relaid. The two men shown in the accompanying view comprised the whole repair gang.

Contracting in the "Dry" States will not appeal to many hearties. The paving specifications of Arkansas City, Kan., (and of other Kansas municipalities, too) contain this provision: "Alcoholic Liquors—All work under this contract shall be executed without the excessive use of alcoholic liquors by the contractor or his foreman. The use of the same shall be considered sufficient cause for declaring the forfeiture of this contract." What constitutes "excessive" use of inebriating fluids evidently depends on the convictions of the engineer. Experience sometimes leads to the belief that alcohol is to a construction foreman what gasoline is to the automobile.

Editorials

Concerning the Miami Flood Studies

The Miami River flood-protection project is so novel—we might indeed say so unprecedented—that there is the more satisfaction in knowing of the elaborate engineering investigations on which it was based. No other construction project in recent years has had as thorough preliminary work done on it.

The engineers faced new problems. They felt it due the problem, due themselves and the people of the valley, that every step in the planning should be questioned and investigated until all chance of doubt was removed. Money enough was available to pay for this investigating, fortunately.

Controlling the floods of a 2,500-mi. area has not been accomplished before; certainly not in America, and nowhere for so flashy a stream as the Miami. It was clear that new means and methods would have to be devised. The starting difficulty, however, was that the data for the problem were lacking.

To begin with, it was necessary to find how great a flood may come to pass in the Miami Valley. Prior storm and run-off studies have dealt with short rainfalls, something radically distinct from the great storms—great in area, duration and intensity—that were in question here. Therefore a new line of study had to be undertaken, flood-storm research and analysis of future storm probabilities. The forecasting, of course, had to be done in the light of what has happened in the past in all parts of the country having similar conditions of climate. To be safe in the forecast it was necessary to search the entire world's records for all pertinent data.

The nature of the flood research, and the reasoning based on it, are sketched in this issue. In subsequent issues of *Engineering News* further problems of the Miami work will be discussed.

The District is planning to publish ultimately its most important engineering studies in the form of separate monographs or reports. Needless to say, these monographs will be valued contributions to the permanent records of civil engineering. They will also furnish the material for a fuller and better understanding of the Miami Conservancy work than can be obtained otherwise. In the meantime, however, the brief sketches of the work given by our present articles afford a more general perspective of the work than has yet been made available to the engineering reader.

It is a fact that outside of the group of men who cooperated in the work of the Conservancy District few engineers have grounds to be otherwise than skeptical and doubtful about the soundness of the project. They know that the scheme contains new elements, but they have not been told why these are necessary, why simpler and better-known expedients would not suffice, or how the Miami engineers have assured themselves that the new expedients will work. Our series of articles will supply answers to these questions. It should suffice to remove the doubts and skepticism. The enterprise deserves to be understood and be believed in by the entire engineering profession.

The work is a monument to the men engaged in it. The main credit belongs to its leader, Arthur E. Morgan, Chief Engineer. But the men in the organization which he brought together contributed knowledge, ability and faithfulness in remarkable degree, and their part in the results is equally worth remembering.

Work for the Successor of the Joint Committee on Concrete

The final report of the Joint Committee on Concrete and Reinforced Concrete, noted in this issue, is an important and praiseworthy document. In a way it marks the end of a structural era for by it the period of doubt regarding the design of reinforced concrete is definitely brought to a close. Through the medium of a score of well qualified experts the developed knowledge of a decade of reinforced-concrete designing has been crystallized so that in fact, if not always in form, it can be used as a standard for some years to come. In the main, the criticisms of the past four years have proved most salutary and the objections to the earlier reports have been considered and satisfied by correction and addition.

It is too much to expect that there will be no opposition to some of the details of design, or that the objectors will invariably be wrong. Universal acceptance of any theory is practically impossible and when theory encroaches so far on the realm of empiricism as does the chapter on the flat-slab floor in this final report, it would be indeed strange if other theories equally defendable did not find their propounders. Such criticisms and objections will undoubtedly come, but as an ultimatum on design the report comes well within the limits of error permitted all joint efforts of this sort.

Design, however, is only a part of the problem of structures, a most important part, to be sure, and one in which it is easy to be didactic. In concrete it happens to be the part which though the subject of more-or-less argument has been the source of the least trouble in practice. One has only to read the stories of concrete failures—complete or partial, sudden or continuous—to realize that the essence of concrete construction is the manufacture of the material and that its future stands or falls upon a better knowledge of that material. And for this phase of concrete the final report of the Joint Committee has little more than the insubstantial generalities of its two predecessors.

Eight years ago these generalities were acceptable. Reinforced concrete was sufficiently novel to make its design of paramount importance and incorrigible optimists were sufficiently in evidence to discourage any serious doubt as to stock methods of construction or to the evil effect of such methods on the permanence of the structure.

Of late, however, whispered doubts have become vociferous. Examinations of concretes of a decade ago have aroused professional fears, questions of aggregate composition and grading absorb laboratory specialists, and the apparently simple matter of consistency has veered from a minimum to a maximum of water and is now swinging back toward the very dry mix of the earlier days. Such doubts and questions, which are the very life of the concrete industry today, could never be learned from the final report of the Joint Committee, unless they could be inferred from the blanket prefatory phrase stating that "there are some subjects upon which experimentation is still in progress."

One cannot justly criticize the committee for not solving these problems. The state of the art probably does not yet permit their solution. It must be criticized for avoiding them, for not, in a footnote at least, making some reference to the doubts that assail the honest concrete expert today in the manufacture of his material, for not referring to such matters as the schism among experts as to the importance of aggregate gradation, the unexplained continuous deflection or shrinkage observed by various experimenters, or to our ignorance of just how much water we should mix for optimum effect.

That such questions remain unanswered by the final report emphasizes the committee's expressed hope that some other equally qualified body will take up the future authoritative study of concrete and that it should not be supposed that all that can be known is now before us. The Joint Committee's arduous work is now done—on its own statement—and well done, but its spiritual successor will have plenty to do.

Making Filter Alum While You Wait

When some Oriental first put a lump of alum in the family water jar to hasten sedimentation he never dreamed of the part that alum or sulphate of alumina would play in the purification of public water-supplies centuries later. Today millions of people throughout the whole world are dependent on the use of this coagulant for clear, sparkling, safe water at a slight increase over the cost of muddy, or color-stained water, perchance teeming with disease germs.

The widespread use of alum in water treatment is due to American brains, energy and money. Beginning and continuing with the remarkable growth of the use of mechanical filters, the use of alum as a coagulant has since been extended in its original domain, or as an aid to clarification by sedimentation—without filters, and also with slow sand filters as well as with mechanical filters. Alum also pays a notable part in the removal of color and even of tastes and odors.

Until quite recently water-purification works depended on the open market for their alum. A few years ago one or two enterprising British cities began to make their own alum—a fact little known in this country. The idea was either taken up or else separately conceived by one of the most progressive water-purification chemists in this country, Mr. Hoover, of Columbus, Ohio, who first worked out a scheme for making syrup of alum at the Columbus water purification and softening works and then turned his attention to the production of crystallized alum. Mr. Hoover's success has been such that his process, now patented, has been put in use at three other water-filter plants in the United States, and one in Canada. It seem destined, like mechanical filtration, to extend throughou the world.

The Hoover process of making alum and its manifol notable advantages are briefly stated elsewhere in thi issue in the introduction to descriptions of alum-makin plants at Columbus, Ohio, Trenton, N. J., and Spring field, Mass. The plant at Omaha, Neb., will be taken u in a later issue. The several descriptions have been pre pared by the chemists or engineers in charge of the sev eral works.

Alum is made regularly at the several works, by means of simple and low-cost installations. It is made up daily—"as you wait," almost—by the ordinary force and under the direction of the chemist or other head of the purification works. The resulting product is not only much cheaper than commercial alum but is also far better suited to use as a coagulant—one reason why it is cheaper.

Particular attention may well be called to the fact that simple as are the process and installation, the descriptions of the four plants show an interesting variety of detail in both design and operation that will repay the study of those interested in the process and also of those who wish to note the many ways there are of solving very much the same engineering problem, so as best to make the solution fit local conditions or personal preference.

Opposition To Postage Zone System

The proposal to increase from two to six times the postage on publications sent over 300 miles, which was reported as a rider on the Post Office Appropriation Bill by the House Committee on Postal Matters has met with strong opposition. The plan of those promoting the bill was to have the House Committee on Rules take action approving the measure as germane to the appropriation bill so that it could be carried through the House as a "rider" with little opportunity for debate. Chairman Henry of the Rules Committee, however, is reported in Washington dispatches as declaring that a strong case against the bill has been made out and that his committee favors full consideration of the bill before passage.

Postmaster-General Burleson has urged on the Post Office committee that in view of the abnormal increase in the price of paper, "most serious consideration should be given to the representations of the publishers and it might be well to consider whether the increase in rates of postage on second-class matter should be made at this time."

A committee representing the editors of technical journals has formulated a protest to members of Congress, a part of which follows:

The chief ground of our opposition is that the effect of this measure would be to destroy one of the strongest forces that has operated to produce national unity. The circulation of national journals has tended to develop common thoughts and ideals in all parts of the United States, and to break down sectional barriers.

Under the zone system the postage on all publications sent more than 600 miles would be three to six times the present rate. The inevitable effect would be the practical suppression of journals of national circulation.

We protest against the enactment of the rider, in addition, on the following grounds:

1. It would discriminate against the farmer, the lumberman, the miner, the merchant, the manufacturer, the physician, the engineer in the remote sections of the country, by charging a high postal rate on information essential to their calling.

2. It would place a crippling tax on the periodical press, which, next to the schools themselves, is the greatest educational power in the country.

3. It would seriously retard our development in agriculture, in trade, in manufacturing, in medicine, science and engineering by restricting the spread of information essential to development in these lines.

4. It would place a drastic tax on the now widely circulated journals of the great scientific and engineering societies, whose members, impelled by patriotic motive and without pay, rendered yeoman service on the Industrial Preparedness Committee of the Naval Consulting Board and which now are giving freely of their time, in answer to the call of the President of the United States, in the work of the National Research Council.

5. It is based on the erroneous assumption that postal expenses increase in proportion to distance. This has been known to be false since at least 1837, when Sir Rowland Hill showed that, even with the crude transportation of those days, terminal expense was 90% of the whole cost of mail service.

6. It would fail to accomplish the object intended, which is to increase postal revenues. The increased cost of subscriptions at long distances would result in a general cancellation of such subscriptions, and consequent loss of the expected postal revenue.

It is to be hoped that the protest made by those connected with the publications affected may be supplemented by many protests from individuals to the Congressmen from their own districts. The Government this year appropriates some $60,000,000 to carry on the rural free delivery service, not because it returns a revenue at all proportionate to its cost, but because the Government desires to bring the benefits of convenient postal service to those who dwell on remote farms. On the same principle, the Government in any revision of the postal rates ought not to take away from the man in remote parts of the country the privilege he now enjoys of receiving the best publications at no higher cost than the dweller in or near the great cities.

A Last Word on Valuation Principles

The Special Committee of the American Society of Civil Engineers on Valuation for Rate Making has presented a final and unanimous report. It is remarkable that seven eminent engineers of such diverse interest and experience as Messrs. Stearns, Metcalf, Churchill, Riggs, Raymond, Snow and Wilgus, should be able to agree so completely on such a comprehensive array of principles and procedure in what has been heretofore such a highly controversial subject not yet emerged from a formative stage. The unanimity of the committee members is especially remarkable in that the report pursues almost to a finality the subject of depreciation, a topic on which committees break up and individuals wax angry. The subject of valuation hereafter ought to be less controversial because of this pronouncement. Until the courts of the land authoritatively settle some pending valuation questions (like the preference for original or present physical conditions, or the treatment of land holdings) and that will be many years, this report must be accepted as a last word.

The report will always stand conspicuous among the reports of the American Society, and indeed among all societies; it probably holds the record for amount of effort involved. All through it are the evidences of the Committee's study, labor, knowledge, wisdom and diplomacy. Yet it is not a report that can be absorbed in one reading, even by those working in this field; it is a document, appreciation of which will come slowly to most readers and the committee has done well to recommend that it be not discussed until the summer convention of the society.

The report is very extended. At some points of its 210 pages it goes to the verge of diluteness, though experience teaches that this will not be an unblessed trouble. Some sections will be more readily grasped than others but eventually the committee's candor and judicial attitude throughout will be recognized; after a few paragraphs have been rephrased, its views, undoubtedly, will be largely accepted. The report will be found pretty generally progressive—it reads the handwriting on the wall even though it has a proper and comforting regard for the equities of property rights developed under a *laissez faire* attitude.

The report in the first place clearly differentiates between valuation for rate-making, purchase, capitalization and taxation. The next effort is to distinguish, for illustrative purposes, between the combinations of principles which should be applied to new properties, old properties under continuous regulation and old properties not under continuous regulation. The report outlines what property is to be covered in an appraisal, and how original-cost-to-date and cost-of-reproduction figures are to be arrived at, leaving it to court or commission to decide which shall have more weight in the ultimate "fair value."

Another point in which the report is diplomatic is the question of whether original or present physical conditions should control in estimating reproduction cost. The committee recommends the assumptions that the identical property is to be reproduced, that present-day conditions affecting cost be considered, and that history be consulted for what is to be considered reproduced and the conditions governing the reproduction estimates. For doubtful or attacked items, it is recommended that the engineer clearly present the results of both theories.

As to the reproduction cost of land, the committee argues for recognition of historical conditions in fixing cost of severance damages, and for transfer of relations between actual acquisition costs of parcels and old normal market value of original tracts to present-day market values of contiguous tracts in valuing carved-out parcels. But the committee displays a caution signal—warning that the engineer should confer with counsel upon the hypothesis most applicable to the case in hand.

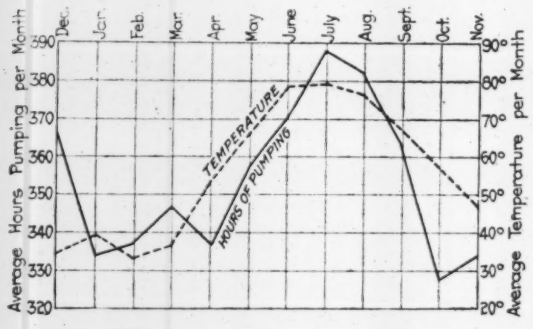
The committee's most skillful work lies in the chapter on depreciation, where it has done the impossible feat of harmonizing the old railroad claim that there was no depreciation of a well-maintained railway in spite of the depreciation of its elements and the reformers' claim that utility rates are to be based on diminished values.

This happy result has been secured by reviving the old word "decretion" and giving it the special meaning of "loss of service life." This it explains is the cause of any depreciation of valuation that can be admitted. Where the public has fully repaid the investor for the cost of this decretion, together with all operating expenses, and a fair return always, the cost of decretion has been converted into depreciation of value, or "fair depreciation" to use the committee's term. Otherwise, in rate-making bases, decretion-cost has been and will be converted to fair depreciation, irrespective of the actual decretion, only to the extent to which the customers have repaid the cost or by which the utility has sacrificed return through laxity, error or shortsightedness. Uncompensated decretion-cost is an obligation of the body of customers—an asset of the utility preserving its equity.

Letters to the Editor

Temperature and Water Consumption

Sir—The accompanying curves show a notable similarity between the number of hours of pumping per month and the average temperature per month for a suburban water plant in Maryland. The plant has five deep-well pumps, 7½ mi. of



TEMPERATURE AND WATER CONSUMPTION

4- and 6-in. main, and supplies a high-grade suburban development. All services are metered, and there is very little leakage from the mains. As information relative to comparatively small water-works is difficult to obtain, it may be that your readers would find this chart interesting.

BANCROFT HILL.

Keyser Building, Baltimore, Md., Dec. 16, 1916.

Baker's Report on Chicago Pavements

Sir—In your issue of Dec. 7 you call attention to the report of Prof. I. O. Baker on his investigation of the paving operations of the City of Chicago, and in particular you quote the amounts of money that he estimates have been lost to the property owners of the city because of defective work.

It may interest your readers to learn of the methods by which he makes these computations. On page 12 of his report he refers to measurements of the thickness of concrete base in three streets and says:

The shortage in each of the three cases respectively is: 21, 12 and 21%, or an average of 18%, regardless of the relative areas or of the relative number of observations. The above are all the definite data I have or can find on the thickness of the concrete base. If it be assumed that these data are representative, the possible loss to the property holders by skimping may be computed as follows: In 1915 the board laid 1,498,000 sq.yd. of pavements having a concrete base. The materials of the base cost about 60c. per sq.yd. Hence the loss was 0.18 × 1,498,000 × 60c. = $161,184.

On page 26 he calls attention to the thickness of the asphalt binder on three streets and says:

In the last three cases the shortage is respectively 23, 27 and 23%, and the average is 24%. I see no reason why we may not assume these examples to be representative. Therefore, since the binder course costs about 46c. per sq.yd. in the wagons on the street, the average saving, as above, is 24% of 46c., or 11c. per sq.yd.; and since about 843,000 sq.yd. of asphalt pavement were laid in 1915, the total loss to the property holders through skimping the binder course was 843,000 × 11c. = $92,730.

He apparently forgets what he has said on page 25:

Actual vs. Theoretical Thickness. Table 2, p. 25, shows a comparison between the theoretical area and the actual area covered by the asphaltic binder sent to three streets in 1915. These data were furnished by the Staff. Notice that the deficiency in thickness varies from 5.3 to 12.3%. In the case of Kildare Ave. the shortage in thickness is equal to ⅞ in. over the entire street, which means that the city got a binder course 1⅛ in. thick instead of one 1½ in. thick, as required by the specifications.

On page 29 he calls attention to the thickness of the asphalt wearing coat on four streets and says:

The shortage in the several cases above is respectively 12½, 12½, 20 and 14%; and the average of all, regardless of the relative area, is 15. I see no reason why we may not assume these examples to be representative. Therefore, since the cost of the 2-in. wearing coat in wagons on the street is about 66c. per sq.yd., and since about 648,000 sq.yd. of 2-in. asphalt pavements were laid in 1915, the total loss to the property holders by the above skimping was: 648,000 × 0.15 × 66c. = $64,152. By a similar computation the loss on 1½-in. asphalt wearing surface was about $14,625. Therefore, the total loss was $78,777.

On page 1 he says:

This report is based upon the following information: (1) Three days spent with an automobile inspecting pavements and asphalt plants: (2) a study of the current specifications of the Board of Local Improvements; (3) a discussion for half a day with the Chief Engineer of Streets from previously prepared memoranda, and correspondence with him since; (4) an analysis made at my request by the Staff of the Finance Committee of the paving specifications of Chicago and of such other paving specifications as were on file in the office of the Staff; and (5) inspection of field books and reports on the data therein by members of the Staff of the Finance Committee.

During the past season the Board of Local Improvements has paved approximately 200 mi. of streets, and the year before we paved less than 150 mi. of streets. It was necessary to increase our inspection force, and skilled competent inspectors were not to be had. It was to be expected that with inexperienced, incompetent inspectors some faulty work would be done. It is, however, not to be expected that a man in Professor Baker's professional position should base conclusions covering 300 contracts from observations made on three or four contracts.

It is of no more importance that an engineer be accurate and honest in making an estimate upon which to base the payment of money than it is to be accurate and honest in making an estimate upon which to base an attack upon another engineer's work.

It is this unfounded estimate of the money lost by the property owners that was seized upon by the daily press as something sensational, with the result that the public was filled with the belief that the Board of Local Improvements and its engineers were corrupt and were robbing the property owners. Even "Engineering News" follows the example in publishing that part of the report which is most unfair, unfounded and defamatory.

There are many things pointed out in this report which call for change and correction, but the character of the report is such as to discourage any reform. Nowhere in the report of 46 pages is there any recognition of merit either in the specifications or in the method of doing work, nor is there any recognition of the difficulties under which the engineering force operates. Everywhere there is fault finding—some of it just, much of it petty, hypercritical or unfounded.

C. D. HILL,
Engineer, Board of Local Improvements.

Chicago, Ill., Dec. 15, 1916.

[In accordance with our usual custom, proof of the above letter was submitted to Professor Baker, who replied as follows.—Editor.]

Sir—The matters discussed by Mr. Hill comprise only a small part of my report, although they are the parts that received publicity in the newspapers. The notices in the newspapers in no way or degree came from me [They are, however, bona fide extracts from Professor Baker's report.—Editor.] and misrepresent the spirit of my report, and say nothing of its constructive features. I do not understand to what these notices refer when they claim that I allege a loss of something like $50,000 (I write from memory) through skimping in the use of cement. IRA O. BAKER.

Urbana, Ill., Dec. 22, 1916.

Badges for Junior Members of the American Society of Civil Engineers are the subject of an inquiry from a subscriber who writes:

Junior members of the American Society of Mechanical Engineers are entitled to wear a society pin similar, except for color, to that worn by the members. Why has the American Society of Civil Engineers never allowed its Juniors the same privilege?

Possibly the reason may be because there are already two kinds of badge worn by those associated with the American Society of Civil Engineers, one a blue enamel worn by Members and Associate Members, and the other a maroon worn by Fellows and Associates. It may have been thought unwise to authorize a third kind of badge.

Court Denies Right To Enlarge Columbus Flood District

The official court hearing on the Scioto River flood-protection plan, drawn up by Alvord & Burdick, Chief Engineers to the Franklin County Conservancy District, in Ohio, began Dec. 6. The hearing covered the two separate questions of approval and adoption of the plan, and enlargement of the district in order to spread the assessment of cost over the whole area benefited by the work. The district as organized is coextensive with Franklin County, in which lies Columbus. The engineering investigation, however, showed that combined retarding-basin and channel-improvement control is best, and this led to application for enlargement of the district. At once the objectors outside of Franklin County raised the legal question as to the right of the district, as at present constituted, to enlarge by taking in the additional territory to protect Delaware, Chillicothe and the farm lands between these cities on the bottoms of the Olentangy and Scioto Rivers.

The argument occupied an entire day. On Dec. 12 the court announced its ruling that the district as at present constituted could not enlarge by taking in territory outside of Franklin County, upon the ground that to take in territory in other counties at a hearing before the judges of Franklin County only would be an unconstitutional proceeding.

This decision nullifies part of the Vonderheide conservancy law. The proper boundary for a large flood protection district such as a watershed of a river or the greater part of an extensive river valley can only be determined after more or less expensive engineering study and investigation. The law provides that a district may be formed upon tentative boundaries by showing in court that protection is needed and can be secured. It provides that the county is the original unit over which a three-tenths mill tax can be spread for the purpose of a preliminary investigation. The law assumes that the funds thus raised will be sufficient to determine more definite boundaries for a proper district in which benefits will exceed costs. After the proposed district has been formed, the official plan approved, and the necessary taxes have been levied upon the district, the money originally advanced by the county is refunded.

The law provides as to how the district shall be extended, namely, by hearings in a court consisting of judges of all the courts of common pleas. The law does not require, however, that the judges of any courts outside of the courts in the originally constituted district shall sit in the proceedings relative to the enlargement or contraction of the district.

A similar question did not arise in the Miami District. There the preliminary expenses were all paid by private subscription, before the original application for forming the district was made. When formed, the district was substantially the same as that affected by benefits and damages of the revised plan finally developed.

The court confined further testimony to costs and benefits to be derived in protecting Franklin County alone, assuming that the works proposed were adequate for the purpose claimed. Hearing upon this question was resumed Dec. 18 and the testimony was completed Dec. 21. No decision has been rendered as yet. If the decision is favorable to the district, it is presumed that the hearing will proceed on the question of the adequacy of the proposed flood protection measures.

Testimony was presented by the district indicating that the official plan as proposed for Columbus and all the territory between Delaware and Chillicothe is the most effective and economical plan even if the entire cost should be spread upon Columbus and Franklin County alone; that the works to be constructed for $9,505,112 include about $2,500,000 for land in the detention basins which would practically be as good for farming as before the construction of the detention dams; that the revenues from this land would be about $120,000 per year; that a water supply for Columbus could be secured up to the year 1950 by the storage of water in the lower ten per cent. of the detention basin outside of the great flood season, thus performing a service to Columbus whose present worth is estimated at $1,000,000. It is expected that the city would finance and acquire by rental this storage right. If so, the cost of the flood-protection would be reduced to about $8,500,000 for assessment against benefited property.

It was shown that the 1913 flood losses in Columbus and Franklin County amounted to $6,176,800 direct loss and about $10,000,000 indirect loss. It was shown by testimony that the real property, including real estate privately held and the real estate of the railroads and public service corporations, amounted to $41,500,000. This is the value as it existed immediately prior to the flood and which it was shown, although materially depreciated, would be automatically fully restored with the assurance of protection. Dividing $8,500,000 into 30 annual instalments as provided by law and deducting the revenue from farm lands in the basins ($100,000 per yr.) the assessment against the benefited real property in Columbus would be about $4.40 per annum per $1,000 of value. This would be about as much as an ordinary paving assessment against a lot. If interest is included (the law specifically excludes it), the assessment would be $7.60 per $1,000.

The objectors presented testimony to show that the value of the real property as shown upon the 1910 tax duplicate in Columbus and Franklin County within the flood zone was about $24,000,000. Upon cross-examination it was shown that this did not include the municipal improvements in the streets nor the real property of the railroads and public service corporations incident to operation, which is carried on the books of the State Tax Commission. These allowances with a further allowance of 25% to represent the difference between the true value of the real property privately held and its value upon the tax duplicate would make the figures on value presented by the district and the objectors substantially agree.

The district presented six prominent real estate dealers who testified that property in the flood zone had been reduced about 50% in value since the 1913 flood, that it would automatically be restored at once with the assurance of adequate protection and within a short time it would be restored to values materially greater than those prevailing before the 1913 flood.

The objectors presented an equal number of real estate men and residents of the flood zone. These witnesses generally agreed that adequate protection would restore values at least to the values prevailing before the flood, but questioned whether the construction of dams would not create a fear preventing restoration of the values.

Average Earnings of Civil Engineers

SYNOPSIS—Abstract of the final report of the Special Committee to Investigate the Conditions of Employment and the Compensation of Civil Engineers of the American Society of Civil Engineers.

The most complete investigation ever made to determine what are the average earnings of men engaged in civil engineering has been completed by a committee of the American Society of Civil Engineers. The report of the committee appears in the December "Proceedings" of the society, and will be presented for discussion at the coming annual business meeting. Two years ago the committee presented a progress report, based on returns showing the earnings of two-thirds of the membership of the American Society of Civil Engineers (*Engineering News*, Jan. 7, 1915). The present and final report includes in addition the information collected from 263 additional members of the society who have joined it dur-

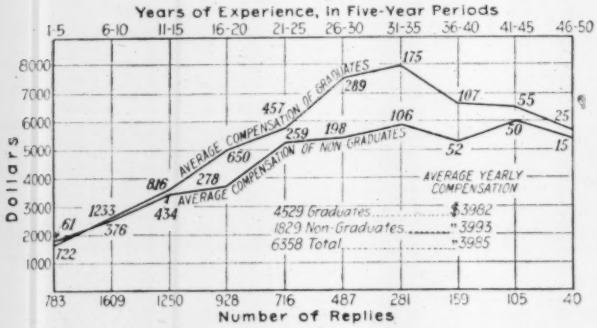


FIG. 1. COMPARISON OF GRADUATE WITH NONGRADUATE ENGINEERS

ing the last two years, and 1,319 reports from engineers who are members of local engineering societies but not members of the American Society of Civil Engineers.

The final statistics now presented, therefore, show the earnings of 6,378 different engineers, representing a total annual income of $25,420,000.

As this journal has pointed out in connection with previous reports of this committee, it must be remembered in studying its diagrams that the figures represent not the earnings of the average men in the profession but of the selected men who reach a position of sufficient prominence to gain admission to an engineering society and to afford its annual dues. Of course, there are a great many civil engineers of high reputation and large professional earnings who are not members of the American Society of Civil Engineers. It will generally be agreed, however, that the average earnings of the 40,000 to 50,000 civil engineers in the United States are very much lower than the earnings of the reporting members of the American Society of Civil Engineers or of the nonmembers from whom the committee received reports.

In studying all the diagrams, it should be kept in mind that the erratic variations on the curves do not represent any general law but merely exceptional variations in the earnings of a few individuals. According to the figures on the diagram, for example, the average yearly earnings of civil engineers of twenty-nine years' experience is $8,690, while the engineers of thirty years' experience earn only $5,431, and the engineers of thirty-one years' experience $8,810. There is of course no reason why the average engineer of 30 years' experience should earn any less than at 29 years. The sudden drop in the curve at 30 years' means nothing more than the men who had just thirty years' experience happened to include an unusual number with very low earnings and few men with very large earnings.

Two diagrams of great interest to the young engineer are those which present the average earnings in various classes of engineering service and in various parts of the country. The very meager salaries paid to engineers in all government work, municipal, state, or Federal, are pretty well known in the profession, and are well shown in the record of average yearly compensation given on one of the diagrams. If these low salaries for public work were compensated for by permanent positions, and by pensions in case of disability or at the attainment of a retirement age, there might be some excuse for these low salaries. Under present conditions, however, the ambitious and competent engineer shuns a position in public service with its uncertain tenure if he can find any promising opportunity elsewhere.

Compared with the salaries paid by cities, states, and the nation, the technical schools show up very well, particularly considering the favorable conditions of service. The railway companies have never paid for civil engineering service what it should be fairly worth, one reason for this doubtless being the very large number of applicants for positions with the railway companies from engineers who have been employed on construction work. The average earnings of the engineers listed as contractors, far exceeds the earnings reported in any other field of work. It must be remembered, however, that the contractor's earnings are always accompanied by a considerable element of risk. In order that the earnings in contracting should be directly comparable with the salaries paid by railway companies or to engineers teaching in the technical schools or to those on government work, a certain deduction should be made for the risk involved in the contracting game.

The diagram showing the variation of civil engineering earnings in different parts of the country puts into definite figures what has heretofore been generally known by those of wide experience in the profession. The best prizes in the profession go to those who achieve sufficient distinction to be sent by American capitalists to foreign countries. New England and the Southern States have the unenviable distinction of paying lower compensation to engineers than any other part of the country, with the exception that a few New England engineers of advanced years and national reputation have very high incomes which serve to bring up the average of those having 30 to 40 years of experience.

A good deal has been published contrasting unfavorably the compensation of engineers and that of other professions, such as lawyers or physicians. Doubtless some of these comparisons are at fault in that the comparison is made between an engineer of average attainments and compensation and a physician who has acquired a large reputation and lucrative practice. The committee in its

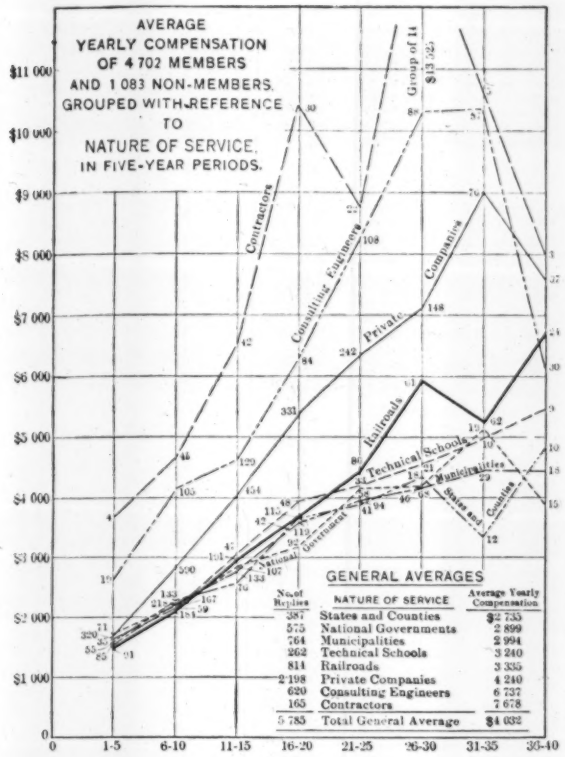


FIG. 2. DIAGRAM BASED ON NATURE OF SERVICE

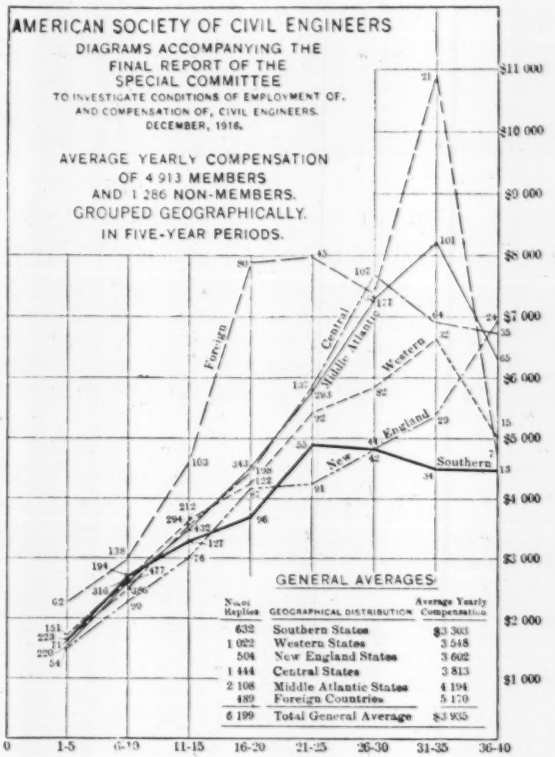


FIG. 3. DIAGRAM BASED ON GEOGRAPHICAL GROUPS

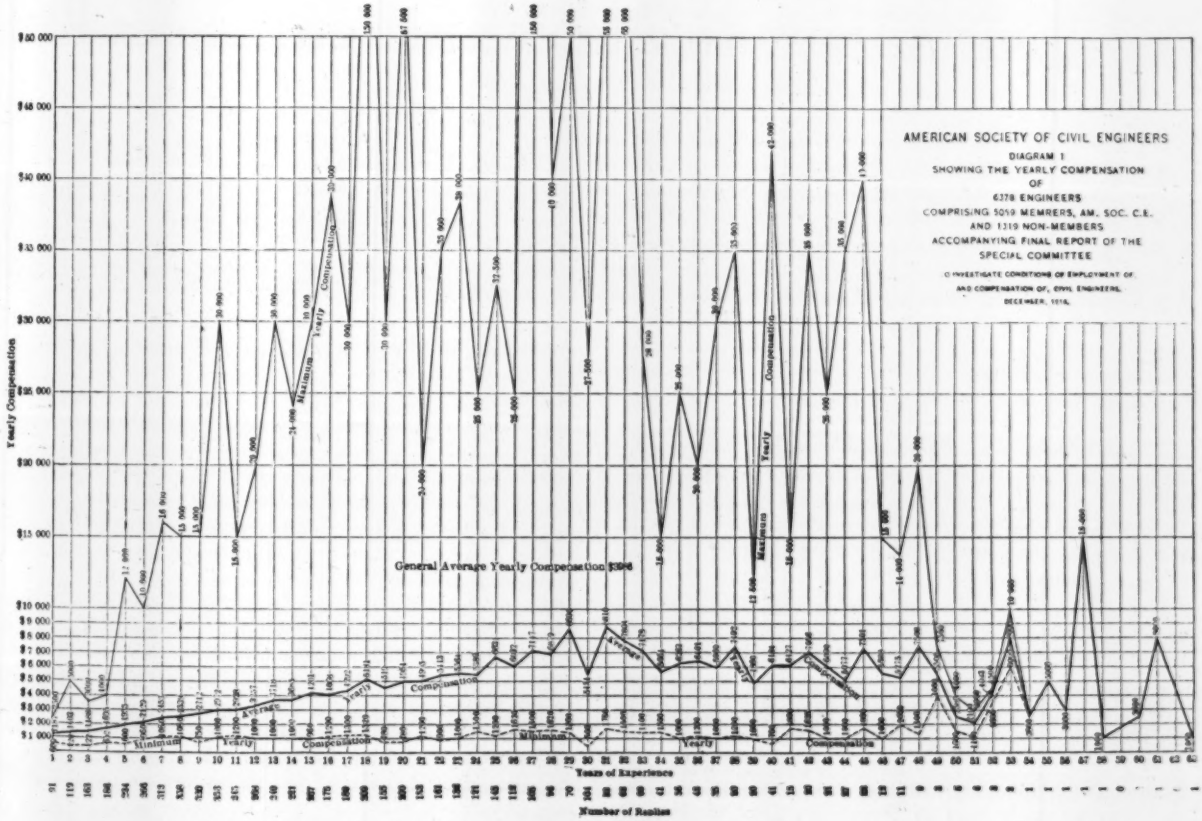


FIG. 4. GENERAL CURVES GIVE INCOME STATISTICS OF 6378 ENGINEERS BY YEARS OF SERVICE

report states that it has attempted to secure information as to the average compensation of members of other professions than engineering, but has not found any definite information. The committee voices its opinion from such meager information as it has been able to obtain that the compensation for engineering work compares favorably with that received by men of any other profession.

When the committee was originally appointed there was an idea that it might make some investigation to determine the causes why a considerable proportion of the engineering profession is underpaid in proportion to its responsibilities and to the expense of professional education. The committee, however, has confined its work chiefly to the collection of the statistics which it presents. The following paragraphs from its report, however, are of some interest in this connection.

The underpaid engineer does not always owe his failure to receive a salary commensurate with the importance or difficulty of his work to the ignorance or lack of appreciation of laymen, but it not infrequently is the fault of his engineering superior in the organization of which he is a part. The replies received by the committee indicate that engineers in private practice sometimes employ men of extensive experience, and presumably of good ability, at salaries which young graduates with little or no experience are able to command, but which are less than those of an ordinary mechanic who has a labor organization behind him.

It may be urged that the competition for work on the part of engineers who employ a technical staff is so keen that it is necessary to take advantage of the needs of those seeking employment in order to secure professional work which is frequently let to the lowest bidder. Engineers in private practice are often criticized because they have not established a scale of charges on a percentage basis as have the architects; but in the offices of leading architects will be found young men who are willing to work for almost nothing for the sake of the experience they hope to gain.

The fact remains that employing engineers are sometimes disposed to pay their men less than their services are actually worth. The code of ethics, adopted by the society by letter ballot on Sept. 2, 1914, lays special emphasis upon the relation of the engineer to his client and to that of competing engineers, who may be striving to secure the same contract or commission, toward each other, but it makes no mention of the obligations of the engineer as an employer to men of his own profession. It appears to lose sight of the need of guarding against underpaying as well as against underbidding.

The engineer, in some branches, unlike other professional men, is frequently unable to locate in one place until he can become known, even though his reputation may be local. He must go where work is in progress, and, when the particular work on which he has been engaged shall have been completed, he must move on to another place. Periods of unemployment are fatal to steady advancement, while a reasonable prospect of promotion, however slow, is likely to prompt men to accept less than the average compensation for the kind of service rendered. . . .

Perhaps there are too many engineers, but the eagerness of young men to adopt this profession is doubtless due to the indisputable fact that young graduates of engineering schools reach a self-sustaining basis, where they are at least able to support themselves, more quickly than do those of other professions.

Much has been said lately about the general decrease in the registration at engineering schools, and this has apparently been viewed with alarm by their faculties. Your committee does not consider it a bad omen for the profession. There is a need of better engineers rather than of more engineers, a need more particularly of men who are better grounded in the fundamentals of the engineering sciences and who have at the same time acquired some knowledge of the economic or business aspects of engineering work, rather than of men who may have the greatest earning capacity on the day of their graduation.

Joint Committee on Concrete Issues Final Report

SYNOPSIS—Salient points in the final report of the Joint Committee on Concrete and Reinforced Concrete.

The long-awaited revised report of the famous Joint Committee on Concrete and Reinforced Concrete has at last been issued. This report, which is definitely announced by the committee to be its final one, had its first publication in the December, 1916, "Proceedings" of the American Society of Civil Engineers as the report of that society's representatives on the committee. It is, however, signed by all members of the Joint Committee, including the representatives of the other societies. The final report is recorded as having been adopted by the committee on July 1, 1916. It will be presented for discussion to the annual Am. Soc. C. E. meeting at New York on Jan. 17, 1917.

It is probably not necessary to outline the history of the Joint Committee. Suffice it to say that it is composed of 28 members representing the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, the Portland Cement Association and the American Concrete Institute. The first report of the committee was submitted in December, 1908. That report was revised and brought up to date in November, 1912, so that the present final report is the third one which the committee has brought in. The final report is signed by all the members of the committee, but Edward Godfrey, one of the representatives of the American Concrete Institute, dissents to some portions of the report, as noted hereinafter.

The main impression given in looking over the report for the first time and comparing it with the 1912 report is surprise at the very few changes. The members of the committee, it is well known, have devoted an enormous amount of attention to the revision. It is evident now that this revision was based entirely on the old report and was really a revision and not a reconsideration of the earlier report. In one notable instance the new report is different; it contains a discussion of the flat-slab or girderless floor construction, a construction that was not touched upon in the earlier report. This is the most radical change. There are, however, a number of other changes each of which has its own importance. The report, incidentally, has exactly the same number of pages as the 1912 report; that is, 52 pages of the regular American Society of Civil Engineers standard.

In the first place, the new report leaves out a great deal of the historical references and authorities noted in the old, and its preliminary statement devotes more attention to the precautions necessary for good concrete work. It makes few changes in the paragraphs relating to the corrosion of the metal, to electrolysis, to the effect of sea water or of impurities on concrete. Apparently, noth-

ing worth while along these lines has been discovered in the four years since the first revision took place. The definitions of materials are practically the same, and the provisions for materials quite little changed. Blast-furnace slag, it may be noted, is specifically not recommended for concrete aggregate, but on the other hand, cinders are allowed in reinforced-concrete slabs up to 8-ft. span. Only the structural-steel grade of the Am. Soc. T. M. billet-steel specification is recommended for reinforcement.

Change In Time of Mixing

In the manufacture of concrete the most definite change is the increase in the mixing period, for a machine mixer, from 1 min. in the old report to 1½ min. in the new for ordinary mixers and 2 min. for mixers of two or more cubic yards' capacity. The need for good and thorough mixing is enlarged upon in some detail. The question of water content is touched upon, but definite instructions are avoided. An additional paragraph has been added on the necessary details of spouting concrete. In the paragraph on freezing weather, the use of salt is deprecated. For underwater concreting the drop-bottom bucket is also deprecated.

There is practically nothing new in the section on forms, and there still is no test given by which one may determine whether concrete has set sufficiently to remove its supports or forms. There is very little new in the section on placing of joints, on shrinkage and temperature changes, on fireproofing, on waterproofing or on surface finish, though all these sections have been rewritten in part.

This brings the report to the subject of design, in which only the important changes can be noted here.

Under certain conditions the brackets are considered in span lengths of beams. The overhang width of T-beams is increased from four to six times the slab thickness. The load distribution on oblong slabs is changed in formula, though the new formula produces practically the same numerical results as the old. In continuous beams there is one minor change of coefficients. The bond-strength and reinforcement-spacing section has few changes. The long discussion on diagonal tension and shear, while rewritten in parts, seems to be mainly a justification of the theory of web stresses which was advanced in the earlier report. Mr. Godfrey dissents from this whole section.

The section on columns has some changes, the first of which is that a definition of a column is changed so that it includes compression members in which the "ratio of unsupported [by which the committee undoubtedly means unstayed] length to least width exceeds about four" instead of as in the old report about six. The second change in the column provisions is that there is no reference whatever made to a hooped or banded column that has no longitudinal reinforcement, and the third change is that the unit stresses in hooped columns (with not less than 1% and not more than 4% of longitudinal bars) and (this is added) with hooping steel not less than 1% of the volume of the concrete, shall have a unit stress 55% higher than for straight-rodded columns, provided the ratio of unsupported length of column to diameter of hooped core is not more than 10 (instead of 8 in the old provision). The old percentage was 45%. This particular clause Mr. Godfrey objects to with the claim that it is capable of being interpreted that hooped columns are given an advantage in the matter of unit stresses only below 10 diameters in height. He recommends a special specification for hooped columns and would not recognize the straight-rodded column at all.

In the section on reinforcing for shrinkage and temperature stresses a paragraph has been added calling attention to the necessity for connecting the various parts of the frame of articulated structures in reinforced concrete. This has undoubtedly been added in response to numerous criticisms of the omission of such provision in the earlier specifications.

Flat-Slab Floor Design Taken Up

The provision for flat slabs takes about eight of the standard-size pages. It is stated in such fashion as to make it difficult of comparison with other used methods for figuring the flat slab. Briefly, the method employed is based on the theoretical considerations laid down by John R. Nichol in his paper entitled "Statical Limitations Upon the Steel Requirements of Reinforced-Concrete Flat-Slab Floors," in the "Transactions" of the American Society of Civil Engineers, Vol. 77. By this theory a certain value is obtained by statical calculations for the numerical sum of the positive moment at a midspan section and the negative moment at a column section of a panel. This value is absolutely determinate. Beyond this point, however, the method becomes empirical. The committee distributes this moment in the proportion of three-eighths to the positive moment and five-eighths to the negative moment. It then further distributes the means for resisting the moment by saying that at least 25% of the positive moment should be cared for in a so-called inner section—that is, an area occupying the space between the two outer quarter-points across the slab. The space outside of these quarter-points, known as the outer sections, should care for at least 55% to 70% of the specified moment. In the negative moment at least 20% should be provided for in the midsection and at least 65% to 80% in the column head section.

In the working-stresses section some minor changes have been made. In the tabulation of the various compressive strengths of different mixtures of concrete, according to the mixture and the kind of aggregate, the proportions of mixture are given as cement to total aggregate and not as cement to sand to stone. The compressive strengths, however, are the same. In the safe values permitted, that for bearing has been changed from 32½% to 35% of the total compressive strength. In axial compression, the short concrete pier has been included. In shear, some minor changes have been made in web reinforced beams. In bond, an additional 1% over plain reinforcing bars has been allowed for the bond stress on "the best type of deformed bar."

The new appendix contains only the formulas, which have some typographical changes from the old. The former cement and steel specifications are omitted, reference being made to the proper standards in other publications.

In conclusion the report says:

The Joint Committee believes that there is a great advantage in the coöperation of the representatives of different technical societies, and trusts that a similar combination of effort may be possible some time in the future, to review the work done by the present committee, and to embody the additional knowledge which will certainly be obtained from further experimentation and practical experience with this important material of construction.

News of the Engineering World

Navy Plans Big Dry Dock at Philadelphia

The United States Navy Department, through the Bureau of Yards and Docks, is asking for bids to be opened Jan. 29, 1917, on a dry dock at the League Island navy yard in Philadelphia, Penn., capable of handling any ship now afloat or under construction. Its main dimensions are as follows:

	Ft.
Length over all, outside dimensions	1,064 3
Length over all, inside dimensions	1,022 0
Width over all, outside dimensions	212 5
Width of entrance, mean high water	132 6
Depth, mean high water	40 0

The structure is to be of concrete, reinforced where required, and is to be founded on piles where conditions show them to be necessary. The estimated quantities are, roughly: Excavation, 860,000 cu.yd.; concrete, 176,000 cu.yd.; piles, 495,000 lin.ft.; structural steel, 120 tons; reinforcing steel, 750 tons; cast iron, 200 tons.

The Philadelphia dock will be the second 1,000-ft. dock on the Atlantic Coast of the United States, the other being the one under construction by the State of Massachusetts in Boston harbor. The United States Government has a 1,000-ft. dock at Balboa, Canal Zone, and the Canadian Government has one under construction at Levis, opposite Quebec. On the Pacific Coast a Government-subsidized dock over 1,000 ft. long is building at San Francisco, and the navy has another under construction at Pearl Harbor, near Honolulu. All of these large dry docks are developments of the last five years and indicate the growth of the large ship, which must have docking facilities at convenient locations.

Relief Work by Engineers in Southern Flood Districts

Engineers everywhere are justly proud that the gigantic task of furnishing food to the starving Belgians was organized and has been carried on under direction of an American engineer, H. C. Hoover. Comparatively few engineers, however, know that the Corps of Engineers of the United States Army, with its staff of civilian assistants, has recently accomplished relief works on a large scale for the benefit of flood sufferers in the South. In July last, great destitution was caused by the severe storms that swept over the Carolinas, Georgia, Alabama, Florida and Mississippi, and $570,000 was appropriated by Congress to relieve the sufferers. Aside from supplying seed to the destitute farmers, which was done through the Department of Agriculture, the relief work was carried out by the Corps of Engineers.

In this work great care was exercised to prevent pauperization. Those applying for free supplies or medical attention were carefully investigated to determine that they were both destitute and unable to work. Most of the relief money was expended by employing the destitute on roadwork in coöperation with state, county and other local officials. Work on the roads was confined almost entirely to grading and drainage done by hand labor, so that practically all the money expended went direct to the workers. The efficiency with which the work was accomplished was highly commended by Senator Underwood of Alabama in a letter to General Black, Chief of Engineers, in which he says: "The work was splendidly done. The relief was immediate and accomplished the results desired without bringing any demoralization in its wake. Our people were thoroughly satisfied with its accomplishment."

Progress on Lincoln Highway in 1916

In New Jersey, where the Lincoln Highway is entirely hard surfaced, $183,678 was spent for repairs and maintenance in 1916. In Pennsylvania the sum spent was $193,000. During the year, contracts were let in Ohio for permanent surfacing in 10 counties, totaling $775,000, exclusive of the paving of the route through many cities.

The counties of Indiana, where there is no state highway department, spent in excess of $700,000 on improvements of the Lincoln Highway in 1916. Incomplete records of expenditures in Illinois total $335,800. The counties of Iowa are not allowed to issue bonds for roads. Consequently, pavement construction is practically impossible and none exists outside municipal limits. Work in the state has consisted of grading, draining and maintaining the route, and in dry weather the Lincoln Highway is a boulevard from the Mississippi to the Missouri. This work cost almost $1,000,000, including the paving of the city streets that are a part of the highway.

Reports from Nebraska indicate that over $367,300, raised largely by the counties, has been expended in Lincoln Highway improvement and maintenance in 1916. The counties through which the highway passes in Nebraska are large and sparsely populated, and sums sufficient to construct hard-surfaced roads are beyond their reach. Yet no difficulties are encountered in dry weather in driving through Iowa and Nebraska; five years ago it was an adventure.

In Wyoming the amount spent on the Lincoln Highway in the past two years has been more than $10 per capita of the population residing along the route. If $10 per capita was raised for road improvement along the Lincoln Highway from New York City to Pittsburgh, Penn., a distance approximately equal to that across Wyoming, the resultant funds would surface the entire Lincoln Highway with concrete from New York to San Francisco twice over.

The Lincoln Highway in Utah, particularly that section lying next to the Nevada line, is the worst section to be encountered anywhere between the two coasts. The improvement of this road presents an insurmountable difficulty to the people of the state. Its cost is far beyond their reach. About the same conditions exist in Nevada, where for 12 mi. in Churchill County the road is through

Fallon Sink, a dried-up lake. For improving this section the Willys-Overland Co. (of Detroit) contributed $50,000, dependent upon the State of Nevada raising a like sum.

A very large part of the Lincoln Highway in the State of California is state highway, and as such has been receiving the benefit of its proportion of the $18,000,000 provided by state bond issue for permanent construction of the state highways. A few sections in California not yet permanently constructed in accordance with the state-wide plan will be taken care of with the money provided through the $15,000,000 bond issue that has just been passed in that state.

Baltimore To Continue Repaving Work

The City of Baltimore, which has been engaged for several years in extensive repaving of a large part of its streets, will continue the work during 1917, spending probably about $2,000,000. Bids for a large amount of paving will be advertised for, about Jan. 15, 1917, the contracts ranging in size from $5,000 for alley work to $200,000 for street work, giving opportunity for both large and small contractors. The city will have available to expend on paving in the next few years about $8,000,000. The work of paving is in charge of the city's Paving Commission, of which R. Keith Compton is Chairman and Consulting Engineer.

Chicago Transit Commission Report

The report of the commission appointed to recommend a plan for rapid transit for the City of Chicago, noted in *Engineering News* of Dec. 7 and 14, was submitted to the City Council on Dec. 21. As previously announced, the commission has laid out a nine-year program calling for an expenditure of $100,000,000 for construction and equipment. In this program is included the construction of a double-track rapid-transit subway 6 mi. in length and a 5-mi. subway for street surface cars, to relieve congestion in the business district. Extensions to the elevated-railway lines of the city will be made, aggregating 65 mi. of single tracks. The network of street railways will be increased by 150 mi. of new line, and 111 mi. of old lines will be reconstructed. Eventually, the rapid-transit subway will have a total trackage of 58 mi.

The commission recommends that a corporation be organized under municipal control to operate all the passenger transportation of the city—surface, elevated and subway—and give universal transfers. Among the first work to be undertaken will be the extension of the present elevated-railway system by the construction of additional lines, and the building of new lines so as to give three through north-and-south routes. There will be one four-track line through the business district and two double-track lines in the western part of the city. On the elevated loop the grades will be separated at the intersection of the east-and-west and north-and-south lines to increase the safety of operation and avoid delays. The present elevated loop will be extended, as the central business district now reaches out beyond the old loop district. As part of the four-track elevated route through the business district a double-track subway is to be built under State St. from 18th St. to Chicago Ave. Other subways will be built to the two east-and-west elevated lines, with a connecting terminal loop. These subways will be at a different level from the north-and-south subways.

At a later date an extension will be added to form an independent north-and-south subway line from 63d St. to Belmont Ave. A street-car subway to relieve congestion in the business district will be located under Michigan Ave., and others will be constructed for the east-and-west line, a connecting loop passing under the north-and-south subway.

ENGINEERING SCHOOLS

Massachusetts Institute of Technology

During 1916, President Maclaurin of the Massachusetts Institute of Technology devoted much effort to the establishment of a larger endowment fund. As a result the total of gifts and legacies secured during the year amounted to $5,796,293. Notable among the items making up this sum is an anonymous gift of $2,500,000, which was conditional on $1,500,000 secured from other sources. The $900,000 legacy for the C. H. Pratt School of Naval Architecture, which sum had been in litigation, was paid. An anonymous gift of $300,000 was made for the establishment of a school of chemical-engineering practice.

University of California

The first of the new buildings provided by the $1,800,000 bond issue, voted two years ago by the people of California, will be placed in service in January. This is Wheeler Hall, a $727,000 white-granite classroom building. By March, the new $1,185,000 university library will be completed; the first part of this, costing $685,000, was built in 1910. During the summer the second unit in the new agriculture group and the first unit in a chemistry group, costing $375,000 and $220,000, will be ready.

NOTES

The Building of Wooden Vessels has not yet ceased to be a considerable industry, according to recent statistics of the Bureau of Navigation, Department of Commerce. There are at present under construction, or contracted for, 116 vessels of over 500 gross tons, making a total of about 160,000 tons.

Track Elevation at Spokane, Wash., by the Northern Pacific Ry., will extend for a distance of 2 mi. through the city. The tracks will be raised 12 to 16 ft., and carried on solid fill between concrete retaining walls, with about 20 bridges over the streets. The passenger station is involved in the change. There are 12 to 15 tracks at some parts of the line and the width of the roadbed will be from 85 to 225 ft. The work will be done without interrupting traffic, which amounts to about 40 regular trains daily. The plan is to build the north wall and then a trestle along the north side from which a fill will be made wide enough for two main tracks. Traffic will then be diverted to these tracks, and the filling continued to the south wall.

Winners of a Prize Contest for articles, written by undergraduates and recent graduates of technical colleges and universities are announced by the Engineers' Subdivision of the Chicago Association of Commerce as follows: First prize of $50 awarded to Harvey T. Hill, Chicago, a graduate of Pennsylvania State College, 1915. The subject of the paper was "Engineering and Civil Progress." Second prize of $30 was

awarded to Leo Shippy, Ames, Iowa, a junior at Iowa State College, who wrote on the subject of "The Engineer of the Future." Third prize of $20 was awarded to H. M. Kistler, Pittsburgh, Penn., a graduate of Pennsylvania State College, 1916, his subject also being "The Engineer of the Future." Eleven universities were represented in the contest. Prizes were offered for the best articles on any one of three subjects as follows: (1) "Engineering and Civic Progress," (2) "The Engineer of the Future," (3) "The Business Relation of the Engineer to the Commercial World." The judges of the contest were, Prof. F. H. Newell, University of Illinois; Dean John F. Hayford, Northwestern University, and John W. Alvord, Consulting Engineer, Chicago.

The Nature of Color in Water was the subject of a somewhat technical but interesting paper by Thorndike Saville, presented to the meeting of the New England Water-Works Association in Boston on Dec. 13. Investigations indicate that most color in water is colloidal in its nature. The author held out the hope that methods of removal better than those now in common use might result from this knowledge.

The Panama Canal Channel through Culebra Cut, according to the Panama Canal "Record" of Dec. 20, is now in better condition than ever before. The cut which has been dredged through the slides has a minimum depth of 33 ft. and the least width of a channel of 30-ft. depth is 180 ft. at the point opposite the rock known as Gibraltar. This rock projects into the channel about 110 ft. and extends along the channel about 200 ft. It has been blasted away until it now rises only about 30 ft. above the water. The dredges "Corozal" and "Paraiso" are at work on the removal of this rock and are dredging behind it to prevent its being pushed forward. Work of the dredging force of the canal on Sundays and holidays was discontinued on Dec. 3, except for the force engaged in blasting at Gibraltar. Since June 1, 1915, dredging has been carried on in the cut continuously with three shifts a day, including Sundays and holidays until Dec. 3.

PERSONALS

S. Camden Miller, Operating Engineer of the Canton Bridge Co., Canton, Ohio, has resigned, effective Feb. 28.

B. J. Simmons has been appointed Division Engineer of the Atchison, Topeka & Santa Fe Ry. at Needles, Calif., succeeding W. L. Bradley, transferred.

W. B. McCabe, M. Inst. C. E., has resigned as Chief Engineer of the Corporation of Calcutta, India, after 13 years' service. He was formerly Water Engineer of Dublin, Ireland.

Kenyon Riddle, recently City Engineer and former City Manager of Abilene, Kan., has been appointed Secretary of the Kansas Automobile Trade Association, with headquarters at Topeka, Kan.

O. A. Tislow, Structural Engineer, has resigned from the employ of Albert Kahn, Architect, Detroit, Mich., to undertake flour-mill building design for the Nordyke & Marmon Co., Indianapolis, Ind.

F. O. Swoboda, Assoc. Am. Inst. E. E., Consulting Electrical and Mechanical Engineer, of Pittsburgh, has been appointed to make a preliminary investigation of the street-lighting system of the city.

Robert H. McCormick, M. Am. Soc. C. E., has resigned as City Engineer of Detroit, Mich., after 16 years in that office. He has been connected with the office of the city engineer for a total of 34 years.

George P. Van Vliet, for the past two years Assistant Engineer with the Division of Sewerage, Cincinnati, Ohio, has resigned to accept a position with the Robinson Clay Products Co., Akron, Ohio, as Sales Engineer.

J. E. Bebb, Assoc. M. Am. Soc. C. E., recently Office Engineer of the Duluth, South Shore & Atlantic Ry., at Duluth, Minn., has been appointed Assistant Bridge Engineer of the Michigan Central R.R., with headquarters at Detroit, Mich.

George C. Hicks, Jr., M. Am. Soc. M. E., for the past 15 yr. Vice-President and Engineer of the P. H. & F. M. Roots Co., of Connersville, Ind., has retired from that company. He expects to take a six months' vacation before engaging in a new enterprise.

G. H. Bayles, M. Am. Soc. C. E., Consulting Engineer, Atlanta, Ga., has accepted a position as Office Engineer of the construction department of the Chile Exploration Co., of New York City, and is now at Chuquicamata, Chile, where his headquarters will be.

Howard G. Borden, a Junior Assistant with the New York Public Service Commission on subway work, has been commissioned Second-Lieutenant, Corps of Engineers, U. S. A., being one of the five civilians who were recently selected by open examinations. He was born in Fall River, Mass., Jan. 13, 1891, and was graduated from the Massachusetts Institute of Technology in 1914.

Harold Ward Sibert, a chemist with the People's Gas Co., Chicago, Ill., a son of Brig.-Gen. William L. Sibert, former member of the Panama Canal Commission, is one of the five civilians commissioned Second-Lieutenant, Corps of Engineers, U. S. A. He was born at Bowling Green, Ky., May 9, 1892, and was graduated from Cornell University in mechanical engineering in 1914.

Thomas F. Farrell, recently with the engineering staff under the Governor of the Panama Canal, is one of the five civilians to be commissioned Second-Lieutenant in the Corps of Engineers, U. S. A. He was born at Brunswick, N. Y., Dec. 3, 1891, and was graduated from Rensselaer Polytechnic Institute in 1912. He was with the Panama R.R. in 1913 and 1914 and since then has been on work in connection with the canal.

Kenneth S. Jones, of the engineering staff of the Illinois Central R.R., has been commissioned Second-Lieutenant, Corps of Engineers, U. S. A., being one of the five civilian appointees, noted elsewhere in these columns. He was born at Norfolk, Va., Feb. 18, 1888, and has graduated three times from the University of Virginia—in arts, law and civil engineering (1915). He practiced law from 1910 to 1913; since then he has been with the engineering departments of the Pennsylvania, Norfolk & Southern and Illinois Central railroads.

Ernest L. Osborne, recently with the Westinghouse Church Kerr & Co., New York City, is one of the five civilians who have been commissioned Second-Lieutenant, Corps of Engineers, U. S. A. He was born in Denver, Mo., June 15, 1889. He was graduated from Sheffield Scientific School, Yale University, in 1912, and after a short experience in engineering work was commissioned Second-Lieutenant, Coast Artillery Corps, U. S. A. He resigned the following year and entered the Massachusetts Institute of Technology, where he was graduated in 1914. Since then he has been an instructor at Yale and a Junior Engineer with the New York Public Service Commission on subway work.

C. K. Clarke, City Manager, of Tucson, Ariz., has been appointed Chief Engineer of the Imperial Irrigation District, Calexico, Calif., succeeding Charles R. Rockwood. Mr. Clarke is a former Superintendent of Water Company No. 1 and Chief Engineer of the Imperial Valley irrigation system under the California Development Co. Afterward he was Division Engineer of the Southern Pacific Co. and was in direct charge of the work of making the closure in the Colorado River break into the Imperial Valley, following the disastrous floods of 1910. He is an intimate friend of Harold Bell Wright, the author, and supplied much of the technical information upon which Mr. Wright's novel "Barbara Worth" is founded. For the past two years Mr. Clarke has been City Manager of Tucson. His experiences here were described in an editorial in "Engineering News," Apr. 20, 1916.

James C. Travilla, of St. Louis, Mo., has been appointed Consulting Engineer of the Dunn Wire-Cut-Lug Brick Co., of Conneaut, Ohio. In March, 1916, he was appointed Consulting Engineer of St. Louis County, Missouri, to take charge of the construction of a $3,000,000 road system. Owing to legal complications the bonds have not been issued, and the position of Consulting Engineer was declared illegal, the courts deciding that the work must be done by the County Engineer. Mr. Travilla is a former Street Commissioner of St. Louis and recently had charge of construction of a $1,000,000 road system for Tarrant County, Texas. He is a graduate of the University of Pennsylvania and went to St. Louis to join the engineering staff of the Missouri Pacific Ry. Later he was Assistant Engineer in the Water Department of the city and subsequently in the Street Department.

Clarence W. Hubbell, M. Am. Soc. C. E., Consulting Engineer, of Detroit, Mich., has been appointed City Engineer of Detroit, by Commissioner of Public Works, George H. Fenkell, to succeed Robert H. McCormick, resigned, as noted elsewhere in these columns. Mr. Hubbell was graduated from the University of Michigan in 1893. From 1898 to 1907 he was Civil Engineer for the Board of Water Commissioners of Detroit. He served in the Philippine Islands in turn as Principal Assistant Engineer on the construction of new sewerage and waterworks systems for the City of Manila; as City Engineer for the City of Manila, and finally as Chief Engineer of the Board of Public Works, Philippine Islands. He returned to the United States in July, 1914, and to date has conducted a consulting practice in sanitary engineering at Detroit. During this period he was retained by the City of Detroit as consulting engineer on preliminary studies for sewage disposal. The salary of the City Engineer is $4,000 per annum.

OBITUARY

Charles W. Archinal, for many years an engineer in the employ of the Trenton branch of the American Bridge Co., died recently. He was a member of the Trenton Engineers' Club.

Peter T. Shelby, a retired official of the Great Northern Ry., who served under Gen. J. F. Casement, contractor for the construction of a large part of the Union Pacific R.R. in the late sixties, died recently in Cleveland, Ohio.

William E. Leard, a manufacturer of New Brighton, Penn., died Dec. 17, aged 74 years. Shortly after the Civil War he engaged in the manufacture of drill presses at Cincinnati, Ohio. In 1882 he built a plant at New Brighton, Penn., for the manufacture of strap joints, connecting rods, forgings, etc.

C. P. Allen, former Chief Engineer of the Denver Union Water Co., Denver, Colo., died from pneumonia at his home in that city Dec. 19. He was born at Moore's Fort, N. Y., Dec. 27, 1842. He went to Denver in 1874 and entered the service of the Denver Water Co., with which company and its subsequent successors, the Denver City Water Co. and the Denver Union Water Co., he remained until 1900. A large part of the construction work done by the original company was under his supervision. After his retirement from the water-works company he entered private practice as a consulting hydraulic engineer and was associated with the construction of water-works at Leadville, Fort Collins, Greeley and Pueblo. He also designed and constructed a water-supply system for the Colorado Fuel and Iron Co. at Pueblo. He served in a consulting capacity for the City Public Utilities Commission in making a report on the cost of construction of a municipal water-works for the City of Denver. In 1912 he was a member of the State Highway Commission. He is survived by two sons, one of whom is William F. Allen, a Denver civil engineer, who has been associated with his father, eight daughters and a widow.

ENGINEERING SOCIETIES

SOCIETY OF AUTOMOTIVE ENGINEERS.
Jan. 11. Annual meeting in New York City. Secy., C. F. Clarkson, 129 West 39th St., New York.

COMPRESSED GAS MANUFACTURERS' ASSOCIATION.
Jan. 15. Fourth annual meeting in New York City. Secy., O. S. King, 120 Broadway, New York.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 17-18. Annual meeting at Society House, New York City. Secy., Charles Warren Hunt, New York.

INDIANA ENGINEERING SOCIETY.
Jan. 18-19. Annual meeting in La Fayette, Ind. Secy., Charles Brossmann, 1616 Merchants' Bank Bldg., Indianapolis.

WESTERN BRICK MANUFACTURERS' ASSOCIATION.
Jan. 20. Meeting in Kansas City, Mo. Secy., G. W. Thurston, 416 Dwight Building, Kansas City.

CANADIAN SOCIETY OF CIVIL ENGINEERS.
Jan. 23-25. Annual meeting in Montreal, Can. Secy., C. H. McLeod, 176 Mansfield St., Montreal.

ILLINOIS SOCIETY OF ENGINEERS.
Jan. 25-26. At Chicago. Secy., E. E. R. Tratman, Wheaton, Ill.

OHIO ENGINEERING SOCIETY.
Jan. 31-Feb. 2. Annual meeting. Ohio State University, Columbus, Ohio. Secy., John Laylin, Norwalk, Ohio.

AMERICAN ROAD BUILDERS' ASSOCIATION.
Feb. 5-9. Eighth National Good Roads Show, in Boston, Mass. Secy., E. L. Powers, 150 Nassau St., New York City.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
Feb. 7-9. Midwinter convention in New York City. Secy., F. J. Hutchinson, 33 West 39th St., New York City.

MINNESOTA SURVEYORS' AND ENGINEERS' SOCIETY.
Feb. 7-9. Annual meeting in Minneapolis.

TENTH CHICAGO CEMENT SHOW.
Feb. 7-15. In Chicago. Under management of Cement Products Exhibition Co., 210 South La Salle St., Chicago.

AMERICAN CONCRETE INSTITUTE.
Feb. 8-10. In Chicago at La Salle. Secy., H. D. Hynds, 30 Broad St., N. Y.

NATIONAL BUILDERS' SUPPLY ASSOCIATION.
Feb. 12-13. In Chicago at Sherman. Secy., L. F. Desmond, 1211 Chamber of Commerce, Chicago.

WISCONSIN ENGINEERING SOCIETY.
Feb. 15-16. At Madison, Wis. Secy., L. S. Smith, 939 University Ave., Madison, Wis.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 19-22. Meeting in New York City. Secy., Bradley, Stoughton, 29 W. 39th St., New York City.

SOUTHWESTERN CONCRETE ASSOCIATION.
Feb. 19-24. Southwestern Concrete Show in Kansas City, Mo. Address Chas. A. Stevenson, 1413 W. 10th St., Kansas City.

The Canadian Society of Civil Engineers holds its annual meeting Jan. 23 to 25 in Montreal. The secretary is C. H. McLeod, 176 Mansfield St., Montreal.

The National Association of Builders' Exchanges of the United States of America will hold its sixth annual convention in Atlanta, Ga., Feb. 13 to 15. The secretary is I. H. Skates.

The Chemists' Club of Youngstown was recently organized with the following officers: President, Edwin G. Pierce; treasurer, H. E. Moyer; secretary, Carl W. Weesner, Carnegie Steel Co.

The Minnesota Surveyors' and Engineers' Society will hold its annual meeting Feb. 7 to 9 in Minneapolis at the Hotel Radisson in conjunction with the annual meeting of the Civil Engineers' Society of St. Paul. Coincidentally the Minnesota Joint Engineering Board will meet.

The American Association of State Highway Officials held its annual meeting recently in St. Louis, representatives of thirty-one states being present. The new officers are: President, George P. Coleman, West Virginia; vice-president, A. B. Fletcher; secretary, J. H. Pratt, North Carolina; treasurer, F. F. Rogers.

The American Railway Engineering Association—The nominating committee has submitted the following nominations for officers for the coming year: President, John G. Sullivan, chief engineer, Western Lines, Canadian Pacific, Winnipeg, Can.; vice-president, Earl Stimson; treasurer, George H. Bremner; secretary, E. H. Fritch, Chicago, Ill.

The American Wood Preservers' Association will hold its thirteenth annual convention in New York City, Jan. 23 to 25 at the Hotel Astor. Among the matters to be discussed are "Service Tests of Ties and Structural Timber," and "Grouping Wood for Preservative Treatment." An informal banquet will be held at 6:30 Wednesday, Jan. 24.

The American Institute of Electrical Engineers meets in Pittsburgh Jan. 12, the subject under discussion being "Braking Electric Vehicles by Regeneration"—that is, using the energy generated on downgrades to apply brakes to the vehicles—by R. E. Hellmund, of the Westinghouse Electric and Manufacturing Co. H. W. Buck, President of the Institute, will preside.

The Society of Automotive Engineers will hold its annual meeting Jan. 11 at the Engineering Societies Building, New York City. Following the presentation of a number of papers on aëroplane, automobile and tractor engines, the society will adjourn to the New Amsterdam Roof to witness "Ziegfeld's Midnight Frolic," the entire seating capacity having been purchased by the society.

The Engineers' Club of Cincinnati held its annual banquet and election of officers at the Literary Club on Dec. 21. A. S. Wilson, Professor of Electrical Engineering of the University of Cincinnati, was elected president and E. A. Gast was reëlected secretary and treasurer. The address of the evening entitled "Concrete Bridges and Viaducts in Cincinnati" was delivered by Frank L. Raschig, the retiring president.

The Florida Engineering Society was organized in Jacksonville Dec. 23, with L. D. Smoot, Commissioner of Public Works, of Jacksonville, President; W. P. Darwin, Jacksonville, and R. E. Chandler, Gainesville, vice-presidents; J. R. Benton, Gainesville, secretary, and H. D. Mendenhall, Lakeland, treasurer. The charter membership list numbers 84, the qualifications for membership being similar to those of the national engineering societies. It is stated that the chief object of the newly organized society is the elimination of objectionable engineering practitioners.

The Florida Association of Architects has congratulated the new Florida Engineering Society, and invited it to coöperate with the association in Tampa, Feb. 8 to 10. The letter of E. A. Ehmann, secretary of the architects' association concludes with: "We shall be glad if you will appoint a committee to meet with us in Tampa, in order to give your and our organizations the much desired opportunity to discuss ways and means to work together harmoniously in our mutual efforts to improve the scientific and practical efficiency of the professions of engineering and architecture."

American Society of Agricultural Engineers—Farm structures and equipment and farm machinery (particularly in regard to tractors) were the leading subjects discussed at the tenth annual meeting of the American Society of Agricultural Engineers. The meeting was held at the Sherman Hotel, Chicago, Dec. 27 to 29, with a registered attendance of over 100 members and visitors. Other subjects included farm planning, drainage and irrigation, and electric power and light for farms. There was also a group of papers on education in agricultural engineering. The President for 1917 is E. H. McCormick, Office of Public Roads and Rural Engineering, Washington, D. C. Secretary, C. K. Shedd, Iowa State College, Ames, Iowa.

Appliances and Materials

New Waterproofing Compounds

A new line of membrane waterproofing compounds is being put out by the Richmond Waterproof Products Co., 30 Church St., New York City, under the trade name "Monarch Cements." Several consistencies are offered, designated as "paint," "x," "xxx" and "xxxx"; the material consists of inert filler, asphaltic binder and more or less oil and coloring matter. These compounds are virtually the older A. F. Roofing Co. cements adopted for a new use, that concern having been absorbed. It is claimed that sufficient color can be introduced to give pronounced reds, greens, yellows, etc., without changing the adhesive and water-repellent qualities of the compound.

The "paint" is used on steelwork where it is desired to protect with a closely adherent fume- and moisture-proof coat. The "x" cement is largely intended for flashing, expansion joints, crack filling, etc.; "xxx" and "xxxx" are to form stiff membranes. All the cements are applied hot, the heavier ones with a hot or greased trowel. Good adhesion to wet masonry, etc., is reported; recent tests in the New York Central Testing Laboratory showed 92 lb. per sq.in.

While the company recommends placing the membrane between the inleaking water and the structure to be waterproofed, yet it claims that the adhesion secured even under the most unfavorable conditions is sufficient so that it stops inflow as an internal, unsupported, plastered-on membrane. The company sells these materials in the open market and contracts for their installation in difficult cases to demonstrate their utility. The paint sells for $1.40 per gal. The cements sell for from 8 to 6½c. per lb., depending on the amounts.

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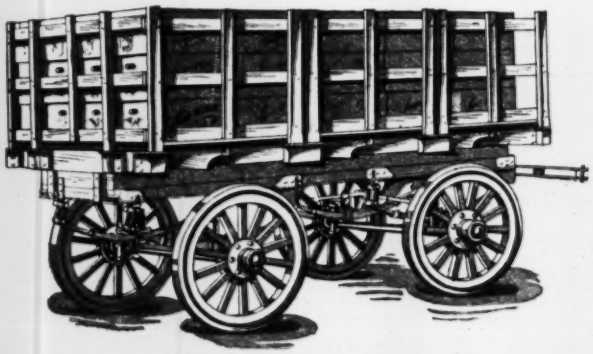
Rubber-Tired Trailer

The destruction of expensive pavements by steel-tired trailers has drawn attention to the field for rubber-tired vehicles of this kind. In the accompanying sketch is shown



NEW TROY RUBBER-TIRED TRAILER

a new design of the Troy Wagon Works, of Troy, Ohio. This was built for the American Brewing Co., of Detroit, and has a capacity of 2½ tons as fitted with an express body. The wheels have 3-in. rubber tires. The trailer is intended for running in only one direction.

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Compressed-Air Meter

A portable compressed-air meter for use with rock drills, etc., has been developed by the Denver Rock Drill Manufacturing Co., of Denver, Colo. It is of positive-displacement type with a piston moving back and forth in a cylinder, the exhaust going to the tool tested. A counter registers the number of strokes; combining the number of strokes, displacement, pressure and temperature gives the amount of free air supplied.

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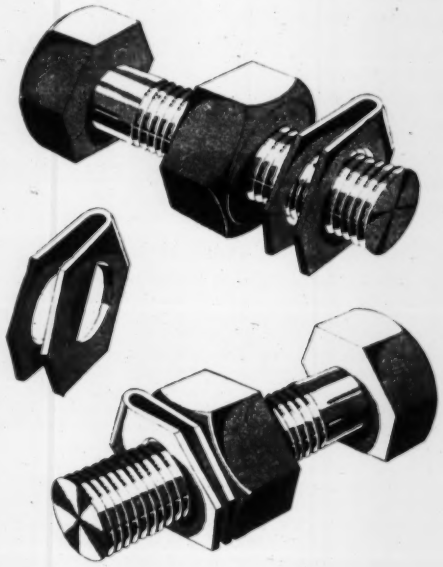
New Spring Nut-Lock

The nut-lock shown herewith consists simply of a steel plate punched with two holes and then bent to a U-shape so that the two holes are opposite but slightly eccentric. This is done in a die press with one operation. When the nut-lock is screwed upon the bolt, the holes are pulled into concentric position, thus causing a direct pull upon the outer leg and a push upon the leg next the nut. This force is exerted at the root of the threads. There is also a lateral force du to the compression of the U-plate when it is screwed hom against the nut. This device is of general application an has been tested with satisfactory results on machinery, au



SPRING NUT-LOCK

tomobiles, locomotives and track bolts. It is being introduced by the Spring Nut-Lock Co., of 608 S. Dearborn St., Chicago, and a contract for its manufacture has been placed with the Stowell Mfg. Co., of Milwaukee, Wis.

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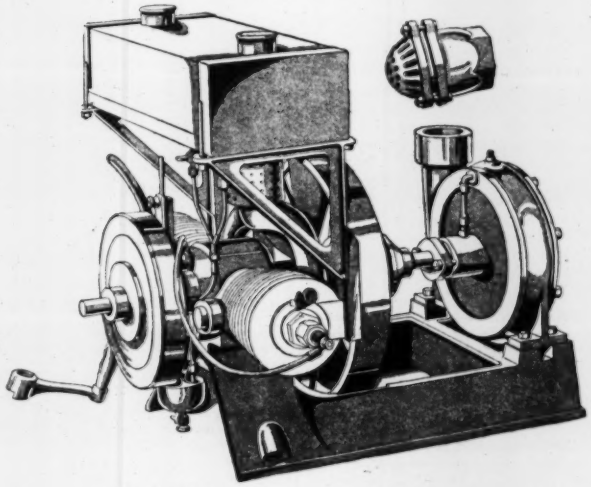
Small General-Utility Gasoline Motor

The "Pormo" portable gasoline engine, formerly sold by the Portable Power Manufacturing Co., of Chicago, is now manufactured by the Aerothrust Engine Co., Laporte, Ind. (with which the former concern has been merged), and is built in two sizes—3 hp., 2½-in. bore, 2¼-in. stroke; and 5 hp., 3-in. bore and 3½-in. stroke. It is of the two-cylinder opposed type and is mounted on a four-legged base. One man can move it about.

This concern uses these engines on special bedplates with 2- and 2½-in. centrifugal pumps, to make handy portable



SMALL CENTRIFUGAL PUMPING UNIT

pumping units. The smaller unit weighs 160 lb., delivers 125 gal. per min. against 30-ft. head and sells for $125; the larger unit weighs 250 lb., delivers 250 gal. per min. against 30-ft. head and sells for $175.